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Engineering News-Record

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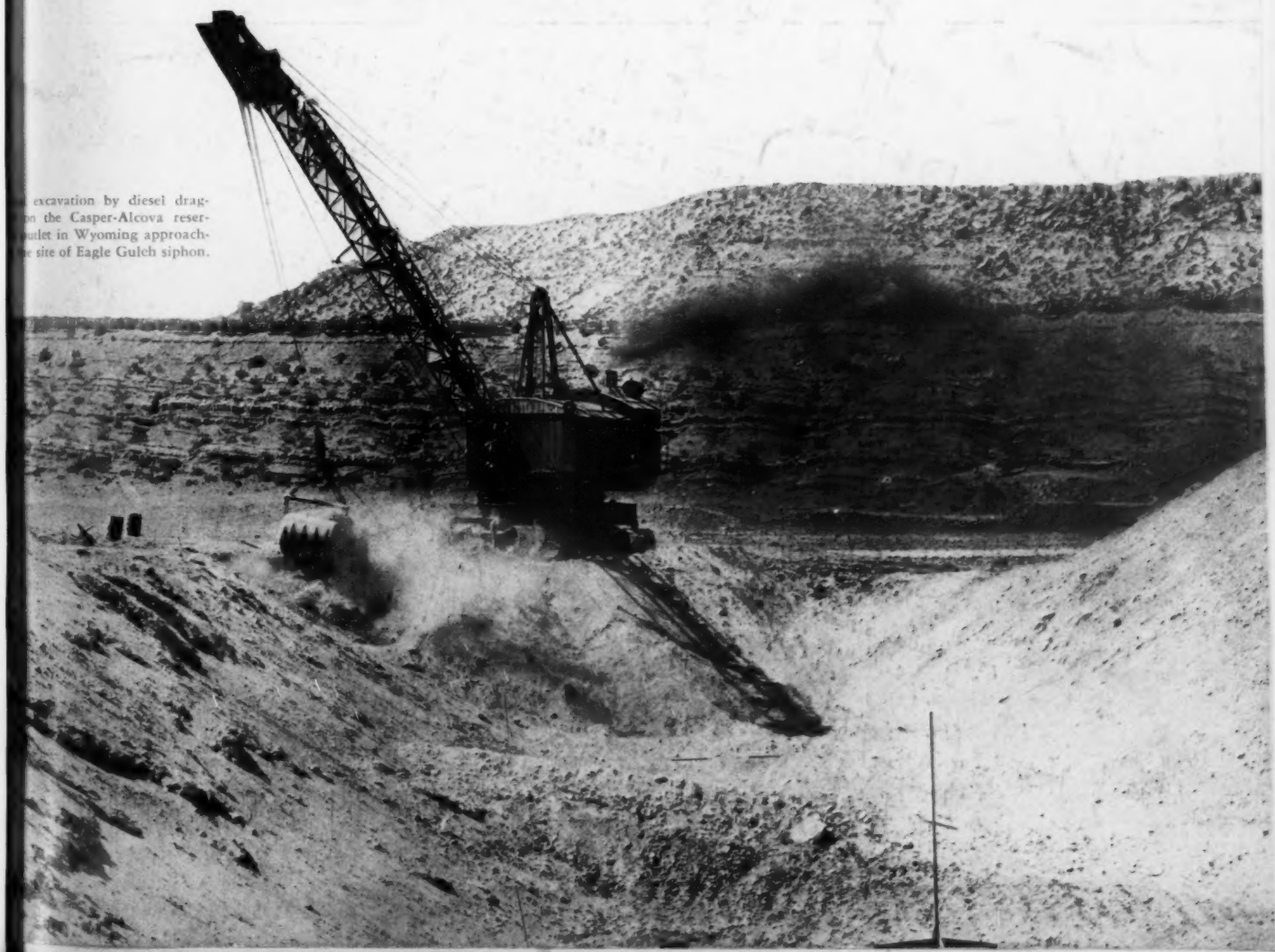
IN THIS ISSUE:

Casper-Alcova Reservoir Outlet

Lessons from the Construction Code

Results of Four-Year Study of Rail Fissures

Welded Bridge Girder Tested to Failure



excavation by diesel drag-
on the Casper-Alcova reser-
outlet in Wyoming approach-
site of Eagle Gulch siphon.

THE SHOW-DOWN ON PREFERENCE

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Fort Peck Dam • Sky-Line Boulevard • Muskingum Valley Flood Project

At Grand Coulee Dam (see photograph above), as on every other big construction job, "Caterpillar" Diesel Tractors stand far in the lead—in numbers, and in performance.

CATERPILLAR

REG. U. S. PAT. OFF.

DIESEL

Why Mess Up Good Jobs?

FROM time to time this page has indicated some of the evils that might result from the unfortunate failure to keep work-relief ideas out of normal public works construction. Object lessons now are coming to hand: one of them is found on a government reservation which for many years has needed extensive improvement and modernization.

Potentially this project is an ideal public works job such as should provide useful employment for the construction industry. It involves works and structures that are badly needed and will constitute a worth-while investment of public funds. It includes a variety of improvements which call for the services of competent contractors and skilled workmen in order to insure not only decent workmanship but also the well-being and safety of those who must live and work on the reservation. To maintain the normal conduct of reservation affairs during construction it is desirable that the various parts of the work be subject to specialized and responsible control.

Accordingly the official in charge of construction prepared plans and specifications for the several classes of work involved, estimated to cost altogether more than $700,000. They cover a number of items that call for special knowledge and skill: several miles of standard high-type pavement with accessory highway structures and granolithic sidewalks; an extensive storm-sewer system; a complete gas-supply system with installations for cooking and heating in all the buildings; further modernization of many dwellings and other buildings, including new plumbing and heating plants, plastering, painting and other finish.

SOME months ago he was ready to take competitive bids from responsible contractors, but along came pressure from above to handle the project on a direct-labor relief basis. Knowing from the nature of the work and the working conditions that such a procedure must be wasteful and destructive, he has done all that he could to resist that pressure. But at this writing he is under instructions to abandon his plans for contract work and to conform to WPA requirements.

This means that he must try to do this work on a basis of $990 per man-year including all materials, although any reasonable procedure would require 2½ to 3 times that sum. The number of men to be employed each week is prescribed and he is restricted to one skilled man to ten unskilled laborers. Thirty hours per week is the maximum per man and wages are to be 55c for unskilled labor, 85c for skilled and 94c for technical and clerical. He is allowed 2 per cent of his labor item for a supervisory force which, however, he can only recommend, subject to approval by the local Democratic committee or by Washington. The work is to be started next month and run through the winter and if workers report for duty but cannot perform because of the weather they must be paid nevertheless.

NEEDLESS to say, this man knows that under such working conditions he cannot produce a job that will be satisfactory either in operation or in quality. But, as in other such situations, the man who is responsible for performance and results and who knows something about the conduct of construction must give way to the influence of a relief administration that knows little about performance and cares less about results; that is concerned only with transferring names from relief rolls to what passes for employment but what actually is a sad messing up of work that should be done by the construction men who need it so badly.

No better example of a truly worth-while public works project could be imagined and there can be no defensible reason for converting it into work-relief. Before this job and others like it finally are denied to a competent construction industry that can do justice to them and perverted into wasteful relief agencies, someone in authority should review the principles laid down by the President to govern the conduct of public works and relief activities.

No real recovery and no constructive reemployment can be fostered by so wanton a sacrifice of both private enterprise and the public interest in the name of emergency relief.

Willard Chevalier

ENGINEERING NEWS-RECORD

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In the News:

WPA WILL GET most of the unallocated $1,250,000,000 work-relief money for quick projects that will carry the unemployed through the winter until the PWA program flowers in the spring, according to Presidential decision. The similarity to last year's decision to use CWA to take up the slack until PWA got started is recalled in most of the news statements on last week's Roosevelt-Hopkins-Ickes conference at Hyde Park.

LOSING NO TIME, the Army Engineers started construction Sept. 15 on the Florida Ship Canal with 100 men, outfitted with "axe and shovel," according to an Associated Press dispatch.

BIDS FOR THE FIRST SECTION of New York City's Sixth Ave. subway have been called for Oct. 4. The construction contemplated is a four-track line between 40th and 47th Sts., where excavation will be mostly in rock.

PARKER DAM having received Congressional approval, which removed Arizona's injunction, is faced with new difficulties. It is reported that the $100,000,000 work-relief allotment to the Bureau of Reclamation does not cover Parker Dam. The bureau is building the dam for the Metropolitan Water District of Southern California, which is a large borrower from the RFC and may now become an even larger one.

In This Issue

THE WATER TO BE IMPOUNDED by Alcova Dam in Wyoming will be led to the main distribution canal system through a series of tunnels, canals and siphons. The interesting variety of construction operations afforded by this 3½-mile-long reservoir outlet line are reviewed in some detail.

STABILIZED-SOIL ROADS, using calcium chloride, are commonly built by road-mix methods. Believing that pre-mixing of the materials at a central plant might be a progressive step, the Dow Chemical Co. set up such a plant as a research unit. The first results of its operation are now made available.

LOAD TESTS on welded plate girders 54 in. deep and 27 ft. long carried out by the highway departments of Kansas and Missouri throw new light on the action of welded structural members. Tested to failure, these girders exhibited properties favorably comparable with similar tests on riveted girders. Particularly noteworthy are the indications of good stress distribution across the welds connecting web and flanges.

CONCRETE BEAMS, reinforced with 2 per cent of steel, have nearly twice as large load-carrying capacity as when only 1 per cent reinforcing is used, according to tests made by the Oregon highway department. Flexural strength is further increased by using greater amounts of steel. Results also indicated that concrete unit stresses could safely be increased to 1,000 or 1,200 lb. per sq.in., as against the usual 650 to 800 lb. per sq.in. specified.

WHAT CAN BE LEARNED from the brief experience of the construction industry with its NRA code? James W. Follin, who was secretary of the Construction Code Authority and followed developments closely from beginning to end of the code system, reviews the facts as a basis for thoughtful study on the part of all those who appreciate that the problems which engendered the code are still alive and must eventually be solved. The first of two articles is published in this issue.

TRANSVERSE FISSURES IN RAILS have been a subject of deep concern to railroad engineers for many years. Methods of detecting them in the track after they have developed had been devised, but little progress had been made toward preventing their formation until an extensive program of research was begun four years ago. What has been accomplished is to be given in a forthcoming bulletin of the A.R.E.A., which is summarized in this issue by Prof. H. F. Moore, who had charge of the test parties.

AMONG THE BILLS passed by the last Congress are a number that are of importance to the construction industry. Summaries of these measures are presented.

Coming Articles:

KANSAS CITY, MO., is ready to open its huge new municipal auditorium, said to be larger than any public hall in the country except that at Atlantic City, which it closely approaches in size. Involved in its construction are some of the heaviest steel trusses and the largest concrete girders yet used in building work. Next week the structural design of this important building will form the subject of the lead article of the issue.

THE SERIES ON RESEARCH at the college laboratories will be resumed next week with an article on the extensive work being done at the Iowa Institute of Hydraulic Research, where unexcelled facilities make both pure and large-scale applied research possible. Subsequent articles will cover the work at Purdue University and Iowa State College.

THE REPORT of the New England Water Works Association convention will be a feature of next week's issue.

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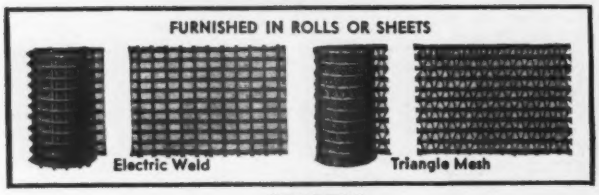

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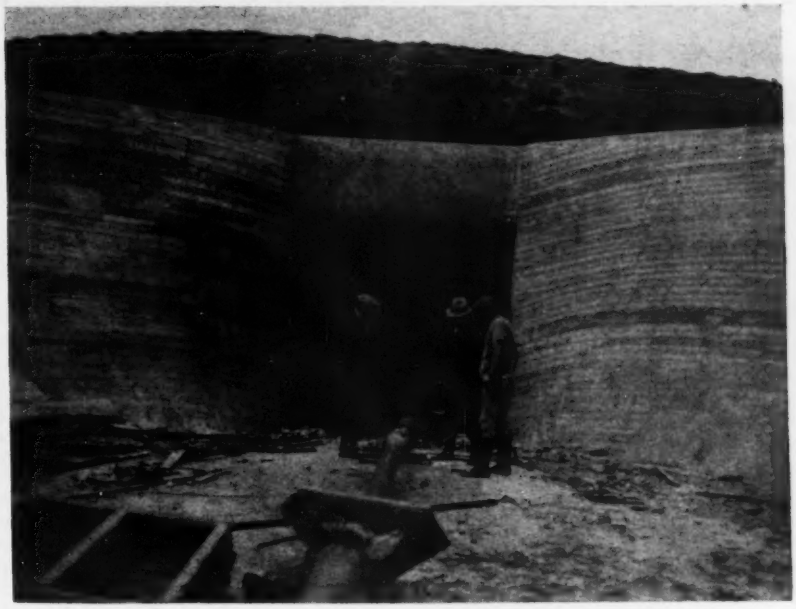


FIG. 1—ENTRANCE TO EAGLE GULCH SIPHON on Casper-Alcova Reservoir outlet works in Wyoming.

Tunnel-Canal-Siphon Outlet of Casper-Alcova Reservoir

Tunnels, open canal and siphons in alternating sequence for a distance of 3.6 miles lead the irrigation waters through ridges to the main-canal distribution system

THE reservoir-outlet works of the Casper-Alcova irrigation system in Wyoming consist of two tunnels through parallel ridges, an intervening canal section with one siphon, and a siphon beyond the second tunnel leading into the main canal system. The tunnels and siphons, with their half-dozen transition sections, make an interesting group of works offering no great construction problems but a nice variety of changes of operation. These are described here. In a previous article (*ENR*, April 14, 1935) construction of the river-diversion tunnels around Alcova Dam was described, and a general article on the whole project was published in *Engineering News-Record*, Nov. 29, 1934.

As indicated by the profile, Fig. 2, the tunnels, siphons and canal form a composite outlet works from the reservoir to the main distribution canal system. Details of the several structures are shown by Figs. 3-5. These structures form the first 3.6 miles of work from the reservoir. Work on this heavy section was started in the spring of 1934 under three contractors, all operating under a single specification, one contractor each for the two tunnels and one for the two siphons, a highway bridge and the open canal.

Tunnel 1—The first headworks tunnel, 2,800 ft. long, takes out through a sandstone ridge about a mile west of and at an elevation 23 ft. below the crest of the proposed Alcova Dam and 13 ft. below maximum high water in the reservoir. The headworks (Fig. 4) consist of a cobblestone-paved flared entrance to a rectangular concrete section, 14 ft. wide, having a radial gate providing an inlet 12 ft. wide when the gate is raised. Back of the gate structure is a 21-ft. transition section leading to the 14-ft. horseshoe section (Fig. 3) of the tunnel. A 12-in. concrete lining with a 7-in. minimum thickness was specified. No timbering was required beyond a few feet at the inlet portal. On account of the steady nature of the tunnel job it was possible to lay out the work on a short-shift basis. During each shift the tunnel progressed 6 ft. Four 6-hour shifts were paid for, but the men averaged only 3½ hours and rarely worked beyond 4 hours.

The material was excavated by an overcasting mucker loading into dump cars hauled out by a gasoline locomotive. Air was blown in through a 12-in. steel pipe by a No. 3 rotary blower driven by a 30-hp. motor. The drill-hole spacing in the 16-ft. unlined face of the tunnel, which ordinarily required

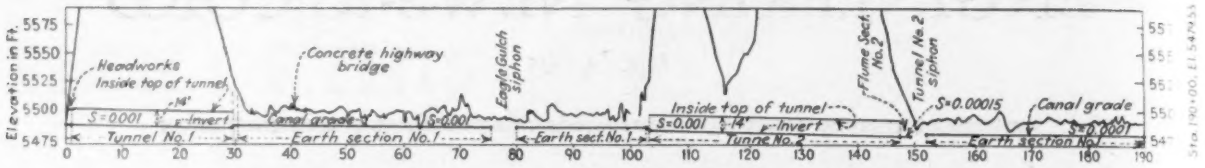


FIG. 2—PROFILE of tunnels, siphons and canals forming outlet works for Casper-Alcova Reservoir

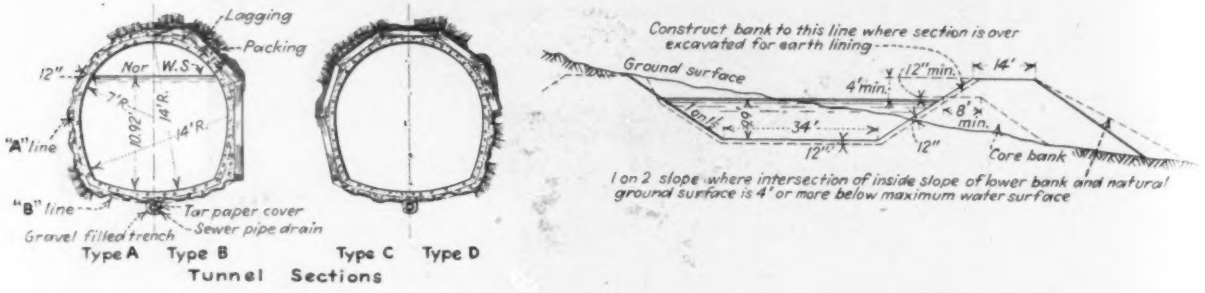


FIG. 3—STANDARD SECTIONS of tunnels and canals connecting Casper-Alcova Reservoir with the irrigation system.

47 holes, was as shown by Fig. 6 and Table 1.

The tunnel was constructed by the Utah Construction Co., which holed through early in January, 1935; this company also lined the adjacent Tunnel 2, acting as subcontractor.

Tunnel 2—Tunnel 2, through a 4,200-ft. ridge, has the same 14-ft. horseshoe section as Tunnel 1, but in it several 50-ft. sections of shale were encountered between the sandstone sections. The shale fell badly and disintegrated on exposure, and here a five-segment arch ring of timbering was used, made up of 10x10-in. native timbers spaced 4 ft. on centers and backed by 3-in. lagging behind which blocks of wood were used as packing in the overbreak spaces.

On shooting, the sandstone was well pulverized into a gravel. In the shale the drill holes were spaced generally about 3 to 4 ft. apart, except in the lifts where the spacing depended on the material, which varied considerably. In the sandstone faces, ordinarily, the number of holes and depths were: five lifters, 12 ft. deep; eight first reliefs, 13 ft. deep; eight second reliefs, 12 ft. deep; eight cuts, 14 ft. deep; one center plug, 8 ft. deep; fifteen rims, 11 ft. deep. These holes were loaded with 262½ lb. of 40 per cent dynamite, which gave an advance of 11 to 12 ft. per round. As the section averaged 8 cu. yd. per foot of tunnel, the dynamite used averaged 3 lb. per yard.

The mucking was done with a crawler-mounted home-made mucker (Fig. 7) with a ¾-yd. bucket of the high overcasting type loading onto a high conveyor discharging into 1½-yd. trucks. The exhaust pipe of each truck was raised above the top of the trucks to disperse the fumes. A 16-in. jute ventilating tube, watertight and airtight, brought in fresh air after blasting. The driving was completed April 8, 1935.

Trimming operation special

A special feature of the work was the use of truck-mounted platforms for trimming the excavation. Details of this work have been furnished by Paul Kistler, foreman for the contractors.

A templet (Fig. 9) was made of 1-in. pipe, welded and braced at all joints with cross-braces. The working crew was composed of six men, two of whom laid the guide track on which the templet was slid along. The contractor also furnished two men to help carry ahead the wedges, blocks, skids, etc., as the templet was used. The skids were 4x4 in. with 2x6-in. plank spreaders, and were set to grade and center. The templet was built so that there was 19-in. clearance. Four members of the engineering staff were used to slide the templet along and do the painting in marking the tight places. The contractor built a platform that could readily be mounted or dismantled from one of the mucking trucks. This was used to stand on, to mark the top and sides that could not be reached from the ground. A driver stayed on the truck and kept backing up against the templet. An extension cord was used to plug in the sockets on the electric line along the side of the tunnel. Three floor lights were fastened to the platform, so that the lighting facilities were very good. The marking crew averaged about 100 ft. per hour actual working time.

A drill jumbo (Fig. 9) was built on a small truck for drilling the holes for trimming. On this were mounted five leyners; three on top, and two on the bottom. Two of the top leyners and the two on the bottom were mounted on bars running lengthwise with the truck, and could be slid along the length of the jumbo for about 12 ft. Then, by using 8- to 10-ft. steel, the drill could be set to reach ahead or behind the truck.

Four drill men and four chuck tenders were used and shifted around on the five drills wherever necessary. One drill on top was mounted on a bar running crosswise of the truck to get the central holes on the top. The three top drills were operated from the deck, and the bottom leyners were operated from the ground. A paving breaker was also carried along with the jumbo and connected to a separate hose, so that spots not big enough to drill and shoot could be knocked off. The water line was taken out as the drilling progressed, but the air line was left in for future use. Tees were installed in the air line about every 200 ft. when it was originally put in.

As soon as the drilling was well ahead, the jumbo was taken out during some of the shifts, and loading, shooting, mucking and other clean-up jobs were carried on. From 300 to 400 ft. were shot and mucked out at a time.

The road grade had been carried at about the top of the invert and, in trimming the first time, just the material that was shot down was mucked out. The engineers then marked the tunnel the second time, and the necessary drilling was done with jackhammers. A platform on a truck was used to stand on to drill the top and sides that could not be reached from the ground. The 4-in. air line was taken out just ahead of the shooting, and a 2-in. line was put in for the drilling the second time. Just before the second drilling was loaded and shot, the invert was scarified with a scarifier pulled with a 60-hp. crawler tractor. By using a man on each side of the mucker to clean the spilled rock away from the

sides of the tunnel, the invert was mucked out fairly clean to solid rock.

As the tunnel was tiled, the high spots in the invert were shot along with the ditch. Where practicable, the high spots were taken off with a pavement breaker. The tile ditch and invert were then mucked out by hand unless an exceptional high spot had been shot, and that was taken out with the mucking machine. The tile was laid on a cushion of gravel, and gravel was placed around and on top. The tile was laid and the tunnel was swept and finally finished right behind the shooting and mucking operations. Thus a good road was obtained for hauling the tile and gravel. As the tile was laid on only one of three eight-hour shifts each day, the following shift would clean up behind and get more ditch ready for the following day.

The engineers took final sections of the tunnel with the templet right behind the laying of the tile. Any tight spots found the last or third time could be taken off with a pick.

By proper organization of forces the loading, shooting, mucking and laying of tile and final clean-up were carried on without any conflict. The loading of holes was done from the same type of platform as was used by the engineers in marking the tunnel. Two of these platforms were built so that any two of the operations for which they were used could be carried on at the same time.

The mucking was done with the mucker built on a 60-hp. tractor and powered by a 40-hp. electric motor. The excavated material was hauled in four trucks with bodies built low, so that they were easier to hand-load than the regular type of body. They were hoisted by means of a mechanical hoist.

The contractor on Tunnel 2 was Edward Peterson. The contractor first excavated 1,200 ft. from the south end until he encountered water, and then moved to the north end.

TABLE I—DEPTHS DRILLED, FUSE DELAYS AND DYNAMITE USED IN OUTLET TUNNEL NO. 1 AT CASPER-ALCOVA

Holes	No. Drilled	Depth Ft.	Fuse Delay	Stocks of 40% Dynamite Used
Cuts	6	14	0	40
First reliefs	8	12	2	35
Second reliefs	13	12	3	15
Rims	12	12	4	12
2 top rims	2	12	5	12
Lifters	6	12	6	40

TABLE II—HYDRAULIC PROPERTIES OF THE TUNNELS, SIPHONS AND CANAL FOR THE CASPER ALCOVA OUTLET WORKS

	A	P	Q	r	n	s
Tunnels 1 and 2	137.44	8.74	1200	4.28	0.014	0.001
Earth sect. 1	483.6	2.48	1200	6.93	0.0225	0.0001
Flume sect. 2	144.37	8.31	1200	4.49	0.014	0.00085

Siphon construction

Two circular 14-ft. reinforced-concrete siphons were used to pass under dry gulches, which at times of cloudbursts or local downpours carry a heavy runoff for a brief period of time. The first siphon, 300 ft. long, crosses under Eagle Gulch, and a second, 220 ft. long, crosses under a small gulch at the end of Tunnel 2. The banks of the gulches for 50 ft. to 100 ft. above and below the siphons were lined with riprap laid on a 1½ slope, to prevent the infrequent floods from cutting back to the portals. The drop in grade of the Eagle Gulch siphon was 16 ft. and of the second siphon 12 ft. The elevation of the original surface was replaced with a final cover over the concrete of 3- to 5-ft. depth. An 8-in. blowoff in a manhole with pipe rising to the surface through a manhole was provided for each siphon.

Because it was only 19 ft. between the outlet portal of Tunnel 2 and the siphon following, which was all in rock cut, a reinforced-concrete flume, 14 ft. wide, was used instead of a canal section. The side walls at the top were 8 in. thick. At the end of the second siphon the outlet transition was laid out on a curve leading into the canal that skirted the ridge downstream.

In building the siphons, which had a minimum thickness of 14 in. and carried 100 lb. of steel to the cubic yard, precast pedestals that acted as anchors were used to support the wood forms. The sections of the siphon were cast monolithic without longitudinal joints, the wood forms, made in 8-ft. sections of sixteen segments, being weighted down on the outside platform with twenty sacks of gravel per 8-ft. section. The concrete was placed by a pump machine from a single set-up for each siphon, the machine being installed in the canal with the mixer on the bank above it and a pipe line leading down the canal to the siphon. Concrete for three 8-ft. sections of siphons was deposited at one time.

Excavation was done with a shovel, dragline and a ½-yd. "tumble bug" scraper hauled by a 60-hp. diesel crawler tractor.

The canal in earth section has a 34-ft. base, ½-on-1 side slopes, a 9.9-ft.

FIG. 4—HEADWORKS and transition sections of Tunnel No. 1.

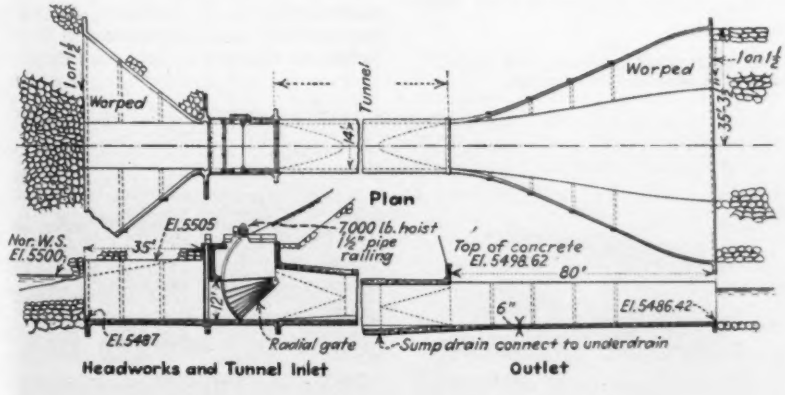
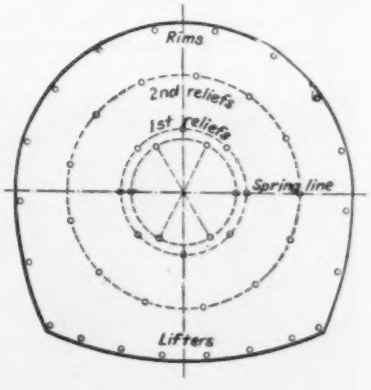


FIG. 6—DRILL-HOLE arrangement in driving Tunnel No. 1.



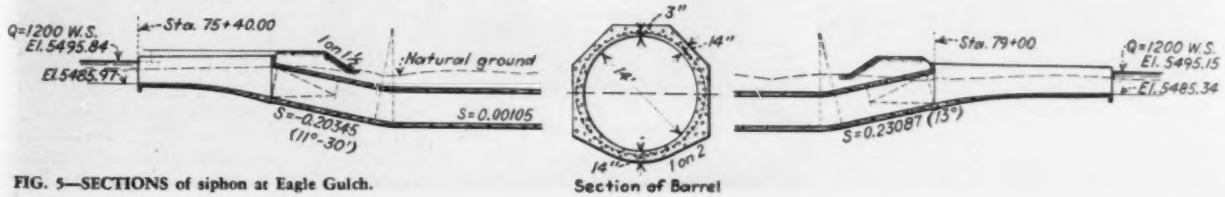


FIG. 5—SECTIONS of siphon at Eagle Gulch.

Section of Barrel

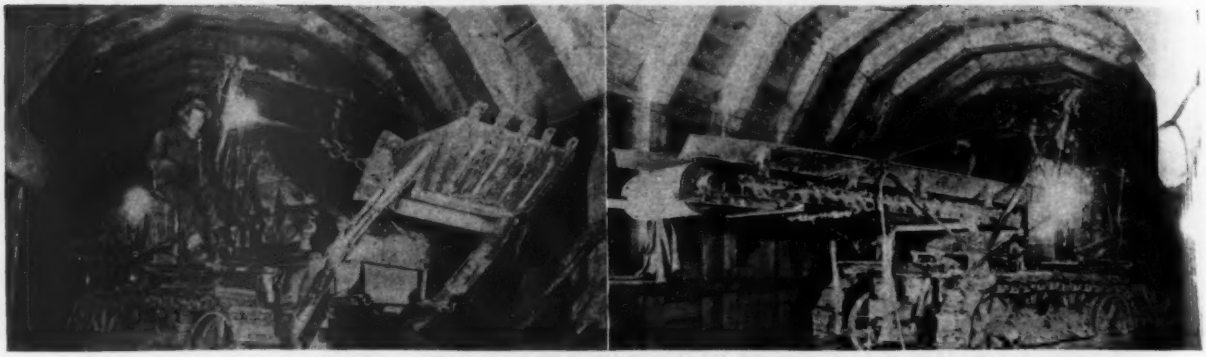


FIG. 7—FRONT AND REAR of mucking machine used on Tunnel No. 2.

water depth and 4-ft. freeboard below the top of the down-side embankment, which has a 14-ft. top width for a roadway. A core bank of fine material, compacted in 12-in. layers, has a minimum top width of 8 ft. and extends at least 12 in. above the water surface. It is placed where coarse pervious material is found. No lining of concrete is contemplated, but where it is considered essential a 12-in. layer of earth is to be placed. The 1,200-sec.-ft. capacity, with the grade of $s = 0.0001$, is estimated to give a velocity of 2½ f.p.s., which in the material encountered will not erode or cut at curves, which are often made sharper. Where shale is found, the cross-section is enlarged so as to permit the use of the fine earth material as lining. Fines are not available in a few places in the excavation and are obtained from borrowpits and hauled in. Overhaul beyond 200 ft. is paid for per yard per 100 ft. of additional haul.

The excavation of the canal is being done with a diesel electric crawler dragline with 3-yd. bucket and 80-ft. boom. The finishing of the top of the embankment and the bottom of the canal is done with bulldozers and "tumble bugs." The embankment top especially is required to be leveled off for a roadway. The deepest cut is 45 ft. through a ridge

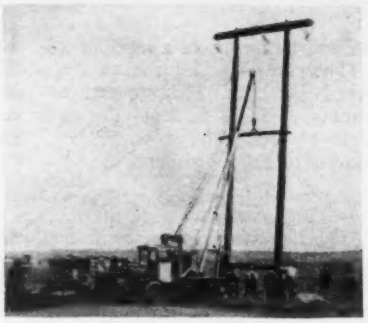


FIG. 8—TRACTOR-TRUCK with tubular gin pole for raising transmission towers on Casper-Alcova project in Wyoming.

below the last siphon. Through this ridge the dragline made two cuts.

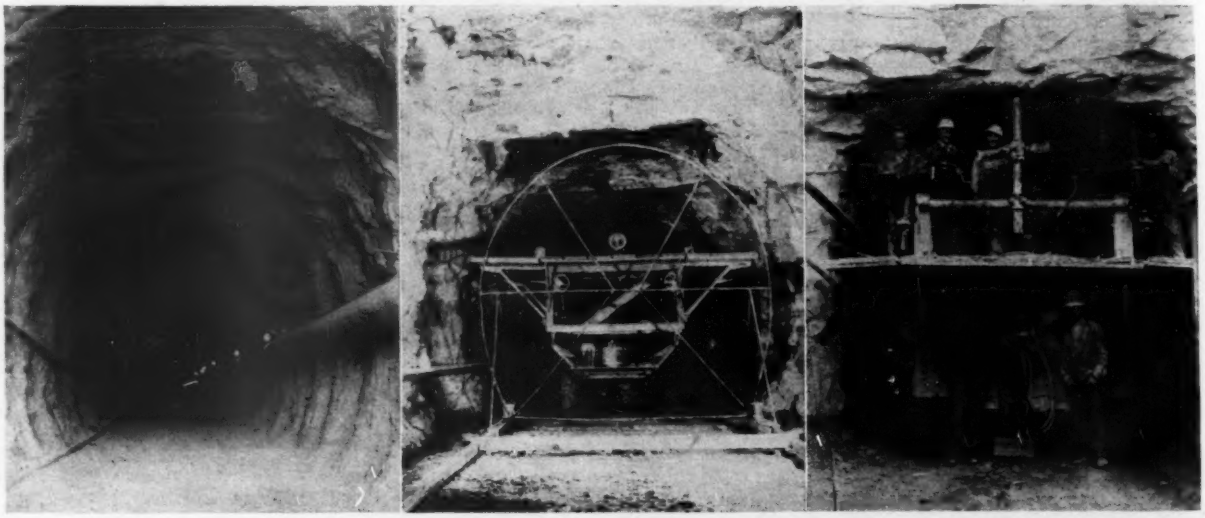
J. A. Terteling & Sons had the contract for the two siphons, the highway bridge and the canal excavation.

Simultaneously with the work on the diversion tunnel and headworks there was carried out by contract a 4½-mile highway of heavy mountain construction into the Seminoe Dam site and a 64-mile 66,000-volt power line from Casper to Seminoe passing Alcova. The highway was completed Nov. 1, 1934, and the power line, started in June by force account, reached Alcova in July and was fully completed within six months. The line consists of aluminum cables suspended from 2-pole H-frames with 20-ft. cross-arms. Erection of the poles, which were framed on the ground, was expedited by a tubular boom mounted on a truck (Fig. 8).

Labor on all of these jobs was obtained from the Wyoming state labor agencies in accordance with PWA regulations. The requirement involved some hardships because few laborers on the agency rolls (many of the cowboy and sheepherder class) were accustomed to heavy construction of this type. It was difficult to obtain experienced construction laborers, and all but foremen had to be recruited locally. By maintaining a camp on the grounds the contractors were permitted to work 40 hours per week, and on some of the jobs four shifts of 5 hours and 40 minutes were maintained per 24 hours.

The Casper Alcova project is being carried out by the U. S. Reclamation Bureau from PWA funds. H. W. Bashore is construction engineer in charge, with I. J. Matthews as office engineer and H. E. Robinson and N. P. Nelson as resident or field engineers.

FIG. 9—TUNNEL 2, showing rock section, templet and drill carriage.



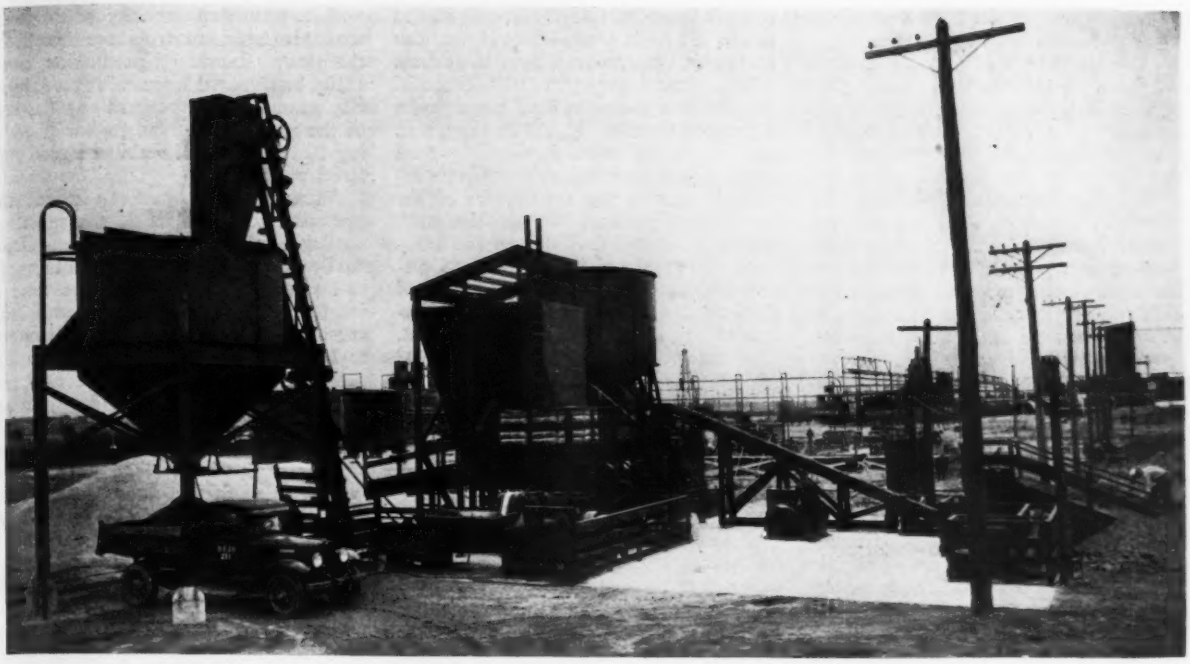


FIG. 1—PREMIXING PLANT for stabilized soil for road surfaces.

Premixed Stabilized Soil for Road Surfaces

Experimentation develops central mixing plant and procedure suitable for road departments or pit owners

By L. C. Stewart and S. J. White
Road Stabilization Dept., The Dow Chemical Co., Midland, Mich.

STABILIZED-SOIL road surfacing is in general practice a road-mixing operation, and current specifications for construction are in general based on this practice. These have been described in various articles and particularly in the articles on "Stabilized-Soil Bound Road Surfaces" published in *Engineering News-Record*, May 24, June 7, 14 and 21, 1934. While an excellent practice in road-mix methods has developed, there has been a growing consciousness that premixing the correct quantities of the proper materials at a plant so that the finished wearing-course composition might be delivered and immediately compacted on the road would constitute a progressive step. It is this reason that has led the road stabilization department of The Dow Chemical Co. to study the subject and report the results of its development work in this article.

At present the demonstration plant for stabilizing gravel, built by the company, has been operated for several months using pea gravel, bank sand, silt, clay and calcium chloride. Slag has been substituted for the gravel during some of that period. The capacity of the plant is between 20 and 25 tons per hour. The pea gravel is shipped in by freight from a producer located about 75 miles distant, and is unloaded into stockpiles at the plant site. When slag was used, it was unloaded directly into a storage hopper. The sand, clay and silt are all obtained on the property of the company.

Plant design

A layout of the plant as finally designed and operated is shown in Fig. 2, while a view of it may be seen in Fig. 1.

The clay and silt mixture is dumped into a spiral feeder from a large scoop scraper after the latter has been drawn through the stockpile and up an inclined ramp by an electric hoist. The spiral feeder is made up of a steel receiving hopper 7 ft. long, 3 ft. 2 in. wide and 3 ft. 6 in. deep. There are four 9-in.-diameter, gear-driven screw conveyors at the bottom of it. These are rotated in opposite directions by two sets of gears, and they carry the mixture to the discharge end where it falls into a hopper over an inclined belt conveyor. An adjustable gate on the feeder can be set in any position to give the desired amount of mixture.

The conveyor elevates the mixture to a hopper that delivers it to a disintegrator, where it is cut into small pieces while passing between a 24-in.-diameter, slow-speed, smooth-face roll and a 12¾-in.-diameter, fast-speed roll, the latter having horizontal cutting knives set into it. Stones up to 1-in. diameter are crushed by the rolls, but the larger stones are thrown out. The clay-silt mixture is then discharged into a crusher that has two 18-in.-diameter smooth-face rolls running at the same speed. There it is further broken up to form a mixture of powder and damp flakes that are about $\frac{1}{16}$ in. thick and may vary from $\frac{1}{4}$ to 4 sq. in. in area. The shaft of one roll is mounted in stationary bearings, while the other has its shaft bearings mounted in sliding boxes held in place by heavy spiral springs that provide for adjustment of opening between rolls. The crusher discharges directly on to the main conveyor belt, which also collects the other materials and delivers them to the mixing equipment. This belt and all other conveyor belts in the plant are 18-in. wide and are the standard rubber type.

A railroad crane is used in filling a 112-ton-capacity storage hopper with gravel from the stockpile or cars. The gravel from this hopper is discharged into a 4-ton auxiliary hopper and then directly onto a belt conveyor that delivers it to the main conveyor belt. A small clearance between the bottom of the hopper and the conveyor belt permits free movement of the belt, and an

adjustable gate at the front end of the hopper controls the rate of feed.

The railroad crane is also used in filling a 45-ton-capacity, rectangular storage hopper with sand from the stockpile. To obtain a free flow of damp sand onto the conveyor belt below, the hopper was constructed with a 60-deg. slope bottom on one side. The other side and both ends are vertical. Several horizontal sliding gates were installed in the bottom of the hopper. These are closed during the filling operation and are opened, one at a time, as the supply of sand in the bin diminishes. The conveyor belt below the hopper discharges onto the main conveyor belt. The rate of feed of the sand is controlled by an adjustable gate at the front end of the hopper.

The calcium chloride is delivered to the plant by truck and stored on an elevated platform adjacent to the 1-ton hopper, which is constructed directly above the main conveyor belt. The calcium chloride is dumped into this hopper by hand, and after passing through a feed control in the bottom it is delivered to the main conveyor belt.

The main conveyor belt delivers the clay and silt mixture, sand, gravel and calcium chloride to a granulator where approximately 6 per cent of water is added, and the combined material is thoroughly mixed. The stabilized gravel is ejected from the granulator onto a short belt conveyor, which delivers it to a bucket elevator that discharges into a 60-ton-capacity storage hopper. From there it is loaded into trucks and hauled to the job or is stockpiled. It can also be loaded into railroad cars if desired.

The granulator that was installed consists of a V-shaped steel hopper with a rounded bottom. It is 7 ft. long, 4 ft. wide at the top and 2 ft. deep. A steel shaft with replaceable mixing blades attached, extends the full length of the hopper and projects through the ends, where it is supported by exterior bearings. The pitch of the blades is adjustable and aids in regulating the rate of discharge and hence the amount of mixing, at any constant speed of the shaft. The latter is operated through gears from a pulley.

Four motors of different sizes are used to operate the plant. The desired speed for each separate unit is obtained by incorporating the correct sizes of pulleys, sprockets or gears. Following is a brief outline of the functions of each motor: a 25-hp., 1,150-r.p.m. motor operates the hoist that hauls the scoop of clay-silt mixture up an inclined ramp to the spiral feeder. A 7.5-hp., 1,150-r.p.m., motor connected directly to a speed reducer, drives the spiral feeder. A 50-hp., 675-r.p.m. motor operates the sand, gravel and clay conveyors, disintegrator and crusher by sprocket and belt drives. The granulator, bucket elevator, discharge conveyor and calcium chloride feeder are run with a 30-hp., 690-r.p.m. motor. The granulator is driven with a 10-in. belt. The elevator, discharge conveyor and calcium chloride feeder are chain-driven.

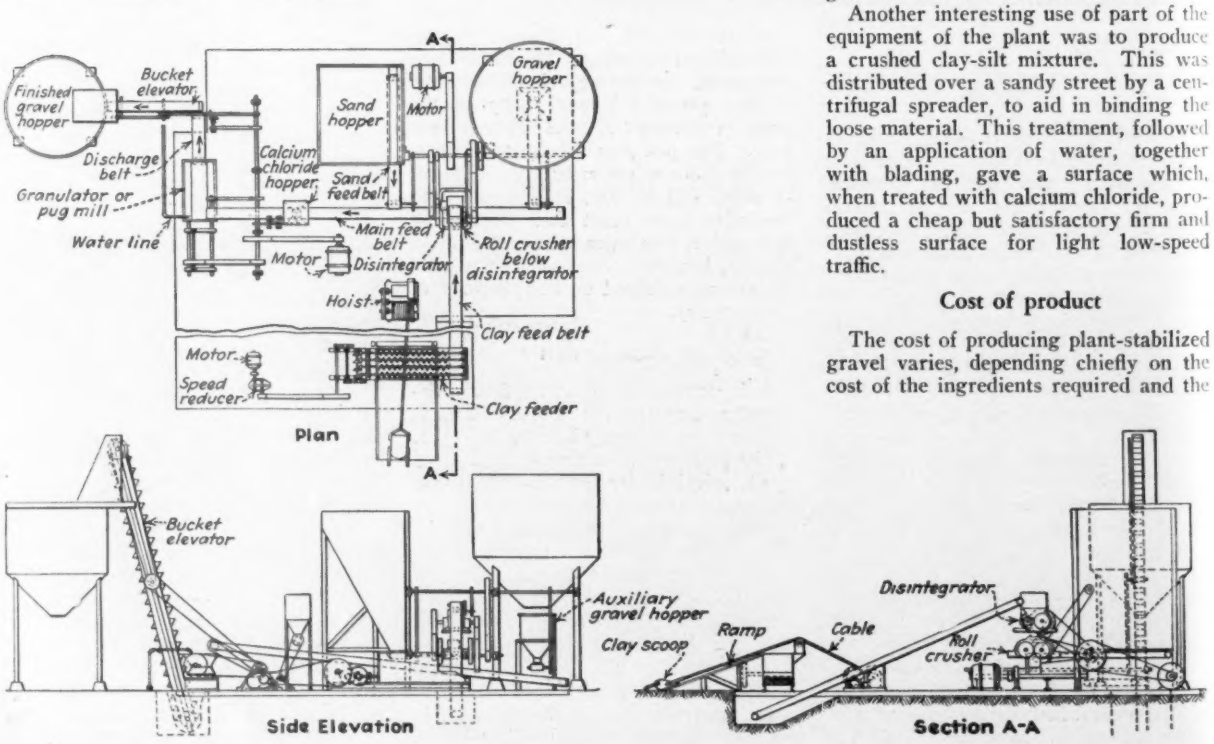
The capacity of the different units varies with the operating speed and size of feed openings. The granulator, with a maximum capacity of 20 to 25 tons per hour, controls the output of the plant. Increased production could easily be obtained by installing a larger-size granulator or pug mill, as the rest of the plant has a capacity for producing 60 to 75 tons of stabilized road material per hour.

Another factor that can affect the operation of the plant is the moisture content of the clay-silt mixture. Ordinarily the pit-run material contains 15 to 20 per cent water. By the time it reaches the disintegrator and roll crusher in dry weather, this has dropped to 12 to 16 per cent. No difficulty has been experienced in using the clay unless it has become quite soggy from rain falling on it while in the stockpile. A cover could be built over the latter, but except for a few long heavy rains the operation of the plant has not been delayed because of lack of shelter for the binder soil.

The uniformity of grading and plasticity index of the product has been very satisfactory. Different shipments of pea gravel have varied somewhat and have necessitated adjustment of sand and binder-soil feeds. In general, however, there has been no difficulty in obtaining a finished material having the proper properties. On one job in which the stabilized gravel was to be applied in a 2-in. layer over a loose floating cover of coarse stones on a road, the sand and binder-soil contents were purposely increased, and the gravel content was decreased. Aided by traffic, a thin layer of this material completely bound together all the loose stones, and the standard product was then applied to give the desired total thickness.

Another interesting use of part of the equipment of the plant was to produce a crushed clay-silt mixture. This was distributed over a sandy street by a centrifugal spreader, to aid in binding the loose material. This treatment, followed by an application of water, together with blading, gave a surface which, when treated with calcium chloride, produced a cheap but satisfactory firm and dustless surface for light low-speed traffic.

FIG. 2—DIAGRAM SET-UP of premixing plant for stabilized soil for road surfaces.



Cost of product

The cost of producing plant-stabilized gravel varies, depending chiefly on the cost of the ingredients required and the

proportions of each needed to produce the proper mix. Some aggregate producers are located more favorably than others with respect to availability of cheap binder soil and sand. In some cases, of course, the aggregate and binder soil will be of such grading that no additional sand will be required. In this demonstration plant about 70 per cent gravel, 20 per cent sand and 10 per cent clay are used, and the total cost of the stabilized gravel, including the calcium chloride, is about $1.25 per ton. This is based on an hourly production of 20 tons. If this rate were increased, the cost per ton would be decreased somewhat. Shipping the gravel from a distance of 75 miles naturally raises the cost well above what would be typical at the plant of an aggregate producer. The table gives a cost summary.

COST PER TON OF STABILIZED GRAVEL

0.7 ton gravel at $1.00 per ton	$0.70
0.2 ton sand at 30c. per ton	.06
0.1 ton clay at 35c. per ton	.035
10 lb. calcium chloride at $19 per ton (in Michigan)	.095
Power	.15
Labor	.10
Maintenance, depreciation, and miscellaneous	.11
Total, per ton	$1.25

Close observation of the roads that have been surfaced with the plant-mixed product has indicated a very dense, firm and dustless wearing course that possessed extra good wet-weather properties.

Ditching by Hydraulic Land Dredge

By A. E. Mosher

Assistant Engineer, W. E. Chapman Organization, Hydraulic Engineers, Miami, Fla.

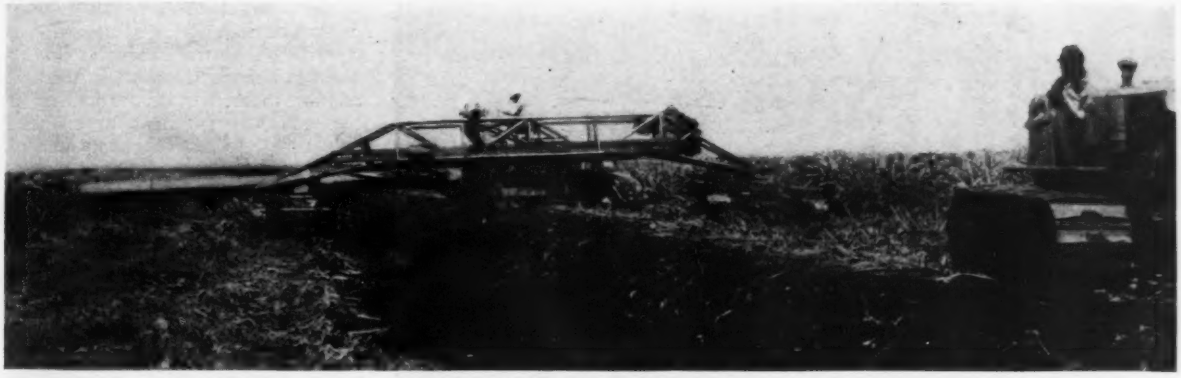


FIG. 1—HYDRAULIC land dredge cleaning drainage ditch on sugarcane plantation in Florida Everglades.

A NEW ditching machine, originating as a ditch-cleaning machine, later used for deepening old ditches and at present being developed to dig new ditches, is now being used by the United States Sugar Co. at their plantation in the Florida Everglades south of Lake Okeechobee. In this country a dense vegetation, principally water hyacinth, and accumulations of fine silt from the muck soil soon clog the ditches and have to be periodically removed. Both hand methods and, in the past, machine methods of clearing have proved expensive, slow and poor in results. With the machine illustrated, some 50 miles of ditches have now been cleared successfully, and the experimental unit has warranted the design and construction of a heavier machine.

The present machine shown in Fig. 1 was designed solely for clearing hyacinths and silt. Fig. 2 shows the casing surrounding the pump casing proper. This casing carries a spiral cutting screw and revolves around the outside of the stationary pump casing. This spiral cutter breaks and cuts the soil and vegetation at the surface and forces it down to the mouth of the pump. It performs somewhat the function of the cutter head on a hydraulic dredge. Both pump and cutter are driven by a 25-hp.

FIG. 2—SPIRAL CUTTER HEAD rotating around suction of turbine pump of Everglades ditching machine.

engine operating on tractor distillate. Discharge of the refuse with a large volume of water avoids any spoil bank, and the fields are left level and unobstructed after passage of the machine. The velocity of flow into the pump intake is such that no volume of fine suspended silt is left to redeposit in the ditch. A clean clear channel to the designed profile is left after the passage of machine. This has been much desired but practically not to be secured with dragline or other means of clearing.

In use at the sugar company it was found possible to deepen old ditches by forcing the head into the new cut and also to dig entirely new ditches, the outstanding virtue again being the entire absence of spoil bank and the perfectly clean channel cut to full profile. In this work it was found possible to operate with as high as 70 per cent solids delivered to the discharge. Removal of material was measured at 3 cu.yd. per minute.

The machine illustrated is the first built; succeeding machines will be considerably heavier and stiffer, to cope with this new use as a digging machine. They will be self-propelled and will not require the tractor shown in Fig. 1. One-man operation will lower costs still more. Driving treads are on fifth wheels and are turned parallel to the body of the machine when moving to a new location. This makes of the equipment a long narrow unit that will handle well and will move in soil conditions which at times are very difficult for man, beast or machine.

Arc-Welded Bridge Girders Tested to Failure

Two 54-in. plate girders, 27 ft. long with butt-welded flange splices, exhibit no distress in welds and good stress distribution among component parts up to point of failure

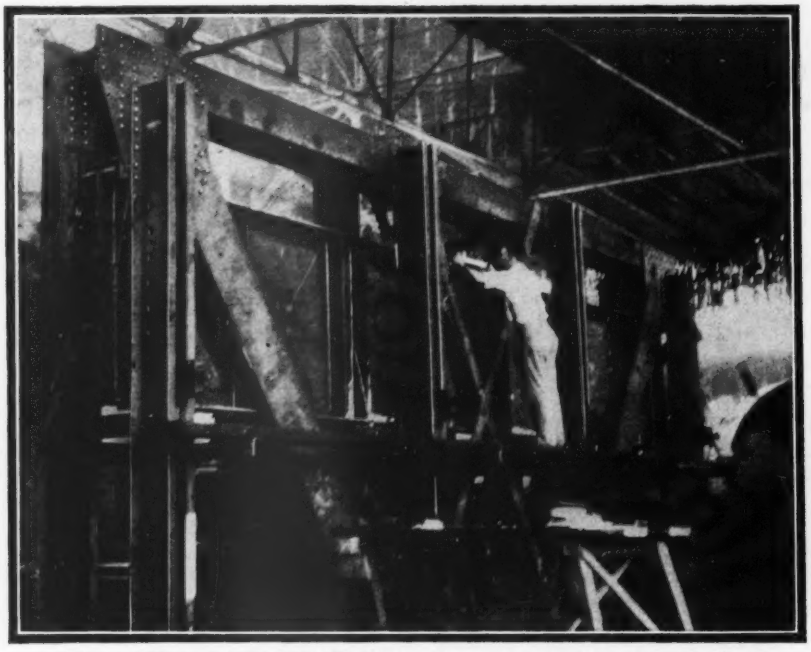


FIG. 1—GIRDER IN TEST FRAME which provided end reactions from 200-ton loading jacks placed near third-points.

By LaMotte Grover
Office Bridge Engineer
Kansas Highway Commission, Topeka, Kan.

TESTS to destruction of two full-size welded built-up plate girders have been made by the state highway commissions of Kansas and Missouri, to gain information and experience relative to the application of welding to bridge construction and also to obtain cost data under shop conditions. These girders, which were composed of 12x1-in. flange plates with butt-welded splices and 54x$\frac{1}{2}$-in. web plates, and which were 27 ft. 1 in. long, were built at the shop of the Kansas City Structural Steel Co. and tested to destruction with two 200-ton jacks in a truss frame (Fig. 1). At various points in the flanges, webs and main stiffeners, deformations were read for the increments of loading, in order to record the general behavior of the girder.

This description is of the test of the Kansas girder, which was a duplicate of the Missouri girder in all respects except that in the Kansas girder half the edges adjacent to welded joints were planed or machined and the other half were flame-cut; also intermittent fillet welds were used for some joints in order to induce higher stresses. Data pertaining to the distribution of stress and general behavior of the girder are included. Some auxiliary tests are being made in the laboratory, which, it is hoped, will aid in the interpretation of the behavior of the welded girder and also indicate the properties of welds that may be expected under shop conditions that prevail in this section of the country.

Every effort was made to make the experiment truly representative of commercial procedure, with the following exceptions: Two makes of shielded-arc-type electrodes were used; one for each half of the girder. One operator handled both makes of electrodes, changing intermittently from one to the other, which is admittedly unfavorable toward uniformity, and would not likely be permitted on regular construction work. Furthermore, the operator was almost totally unfamiliar with one of the electrodes. Also, the work was rushed to completion, so as to release parts of the equipment that were available for only a limited time.

Butt-welded splices were provided in both the tension and compression flanges in order to test this type of joint. The preparation and welding procedure of these joints is shown in Fig. 3. The web was also spliced at two points by introducing a 12x$\frac{3}{8}$-in. plate, which acted as a stiffener for both sides of the web and at the same time provided for making a fillet-welded web splice. One web splice was made with the sequence of welding that would be used for making a shop splice, and the other with the sequence that would be necessary in the case of a field splice (Fig. 3). The flanges and web were each built up separately before being joined together, and in each case an extra length of material was provided and a sample joint prepared and cut out for an independent test piece, thereby providing the same chilling effect for welding the sample as for welding the regular splice.

After the component parts had been completed, one flange was leveled on the floor, the web plate placed on it in the proper position and tacked to it throughout. The web splice stiffeners, which were tacked to the flanges at this time, aided in holding the parts in the proper relation to each other. The weight of the web was sufficient to enable the flange plate to be brought up into close contact by wedging under it until the tack welds had been made. The other flange plate was then leveled on the floor, the web placed upon it and tacked in the same manner. Care was taken to place tack welds at each point where the stiffeners would intersect the flange because these points would be inaccessible later.

The stiffeners were next tacked in place, after which the flange-to-web welds were completed. A "step-back" sequence was employed, to avoid distortion. The use of only one welding operator was found to cause some inconvenience. Under ordinary circumstances two operators would be employed, working on opposite sides of the girder, starting at the middle and progressing toward the ends, possibly using a step-back sequence.

All of the fillet welds were $\frac{1}{4}$ in. except in the web splices, where $\frac{1}{2}$-in. welds were used. All welds between the stiffeners and the web and flanges were made in the flat position, which increased the speed of welding but necessitated some additional handling. Although the stiffeners were attached after the flange-to-web welds were completed, it is possible that the better procedure would have been to weld the stiffeners to the web first. The latter procedure has been found to be the best by some German investigators.

Test results

In addition to reading deformations in the various parts of the girder, both vertical and lateral deflections were measured at eight points in the compression flange. The vertical deflections are plotted in Fig. 4 against the load on each jack. The yielding of the girder

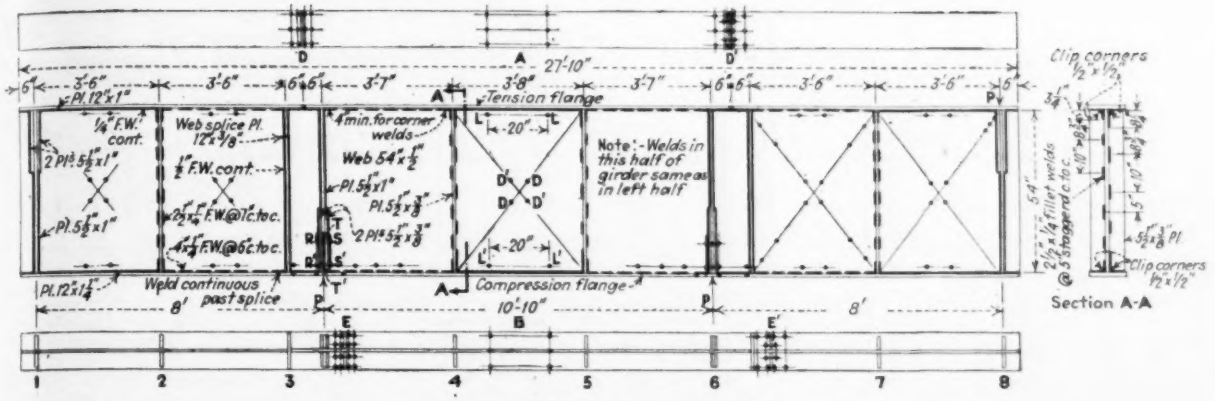

Tension flange
Compression flange
Web splice Pl. 12"x 3/8"
4 min. for corner welds
Web 54"x 1/2"
Note:-Welds in this half of girder same as in left half
Weld continuous past splice
Clip corners 1/2" x 1/2"
Section A-A
Clip corners 1/2" x 1/2"

FIG. 2—WELDING and gage-point details of plate girder that was loaded to failure.

was so gradual throughout the entire range of loading that no definite or sudden yield point could be established. In this respect the behavior was similar to that of riveted girders that have been tested by various laboratories. Based upon the criterion of the yield point being at the point where the slope of the tangent to the deflection curve is twice the slope within the elastic range, the yield point would be considered as occurring at a load of about 230,000 lb. on each jack, closely checking the theoretical yield point at a load of 260,000 lb. on each jack required to produce a stress of 32,000 lb. per sq.in. in the tension flange. There are no indications of a definite yield point in any of the strain curves for the component parts. The safe design load for a basic stress of 18,000 lb. per sq.in. was calculated as 165,000 lb. on each jack. The actual ultimate load can be considered as approximately 320,000 lb. on each jack. It appears that had the girder been supported under a free static weight of this amount it would have collapsed through failure resulting from elastic instability.

Although failure took place ostensibly as a result of the elastic instability of the compression flange, nevertheless deformations measured in the tension flange indicate that the ultimate strength would not have been increased a great deal by further restraining the compression flange from lateral deflection. Lugs or lateral jacks were provided at points near the loading jacks, but these were not used to any great extent because they were not designed properly to permit freedom of vertical movement.

Interpretation of results

The deformations read with the strain gages were all multiplied by 30,000, to enable comparison with theoretical stresses within the elastic range. The sections of the girder were not calipered for computing theoretical stresses.

In interpreting the deformations that are recorded in the accompanying diagrams (Fig. 4), it is necessary to take into account the tendency toward lateral bending in the compression flange throughout almost the entire range of the test, and uneven stress distribution caused by the lateral bending must not be misinterpreted. No yielding due to vertical loads except that of a local nature was observed at any points in the girder except in the flanges at the center of the span. The way in which the deformations in the web adjacent to the flanges at these points follow those in the flanges so closely illustrates the homogeneity and good stress distribution that is attained with welded construction.

Although stresses considerably above the yield point were indicated adjacent to the butt welds, the maximum stresses observed at the 1-in. gage points across the butt welds were well below the yield point of the weld metal. This is to be expected under a static test of this kind because the yield point of weld metal is considerably higher than that of structural steel. The Huggenberger tensometers installed at one tension flange and one compression flange joint (joints D' and E') yielded good consistent readings, but a damaged strain gage used for the 1-in. gage points in the other welded flange joints failed to give intelligible results.

When lateral bending and distortion of the girder became severe, some of the fillet welds were stretched to a considerable extent, but none of them cracked even under these severe conditions.

One side of both the Kansas and Missouri girders was whitewashed. Excellent patterns of the strain lines appeared on the Missouri girder during the test, but very poor results in this respect were obtained for the Kansas girder because the whitewash was not prepared and applied properly.

Lateral bending became quite severe after the load of 260,000 lb. per jack (theoretical yield point) was passed, and consequently deformations as read in the compression flange are without significance for loads beyond this point. Attention is also called to the fact that the compression flange bent in the shape of a reversed curve because of the lateral support it received from the jacks as the ultimate load was approached, and therefore the effect of lateral bending is not in the same direction throughout the length of the girder.

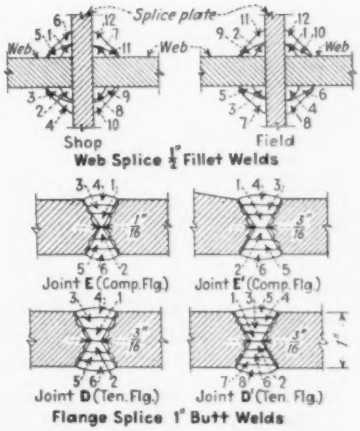


FIG. 3—WELDING SEQUENCE used in making splice joints in web and flanges of test girder.

It is interesting to note that the vertical stress in the web of the girder directly over each jack and adjacent to the main stiffeners is about half as great as that in the outer edges of the stiffeners.

There was no evidence of any deformation caused by diagonal tension or compression across the gage points DD and $D'D'$ at the center of the web until it began to bulge.

It is evident that no part of the original failure is due to excessive stress in the web, which was adequately reinforced with stiffeners.

In general, the stresses calculated from the deformations agreed quite well with those computed for the loads; and the apparent yield point and ultimate strength as noted during the test agreed very closely with those observed in the test of the Missouri girder.

Auxiliary tests

After the girder was tested, each of the butt-welded joints was cut out from the flanges; likewise spliced portions of the web were removed. These pieces

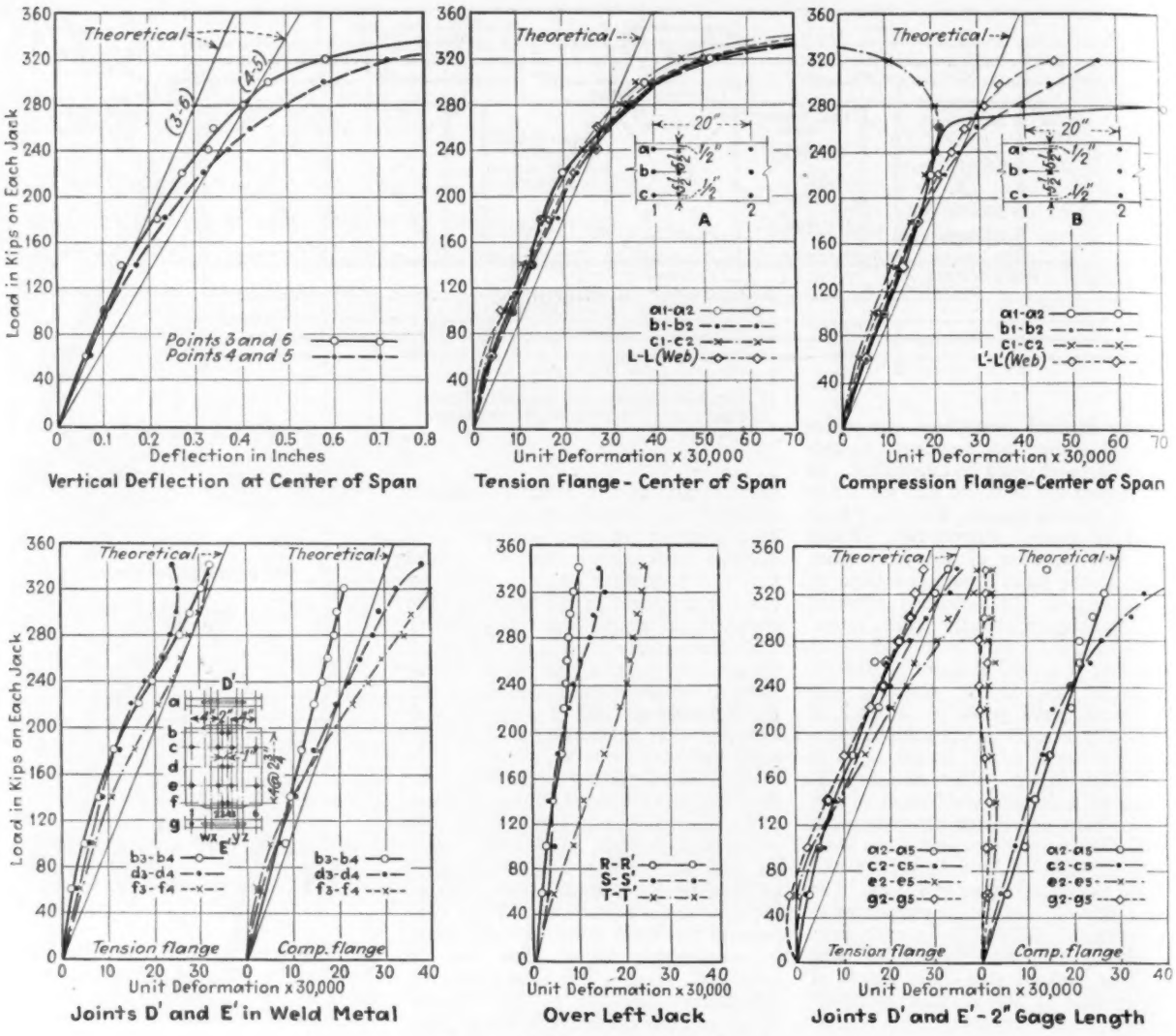


FIG. 4—STRAIN-GAGE and deflection curves for representative points on the girder. The good stress distribution shown is typical of what was recorded for the other gage points indicated in Fig. 2.

as well as the unstressed sample flange and independent web-splice test pieces were cut into strips, to provide specimens for laboratory tests, which will include tension tests in which deformations will be read from zero stress up to the yield point, notched-bar impact tests, tension-impact tests and possibly repeated-stress tests. By comparing strain-gage readings on the tensile test specimens with those on the joints of the welded girder, it is hoped to obtain some information relative to residual stresses caused by welding.

The evidence of the soundness of the welds that was obtained in cutting up the specimens is one of the most valuable parts of the information gained from the experiment. When a gas cut is made across a welded joint, it is very easy to detect the presence of particles of slag that lie in the path of the cut, because even a very small particle will cause an obvious irregularity in the edge of the metal. Very few instances were observed where defects existed in the welds, and those noted were very small in size and can be attributed for the most part to unnecessarily severe restrictions upon the use of a diamond-point chisel in cleaning the weld layers previous to depositing subsequent runs, and also to improper treatment of the ends of the butt welds.

Acknowledgment

Special acknowledgment is made of the assistance of W. E. Gibson, engineer of tests for the Kansas highway commission, whose supervision of the drilling and reaming of the gage points and the reading of the strain gages is responsible for the excellent results obtained; also of the cooperation of Harry Darby, director; W. V. Buck, state highway engineer at the time of the tests; O. J. Eidmann, engineer of design; and H. Allen, engineer of materials; also of the courtesy of N. R. Sack, bridge engineer of the Missouri highway commission, in permitting the use of the testing frame that was first constructed for their use; also of the cooperation of the Kansas City Structural Steel Co. in performing the work without allowance for overhead expenses and profit, and that of the General Electric Co. and the Lincoln Electric Co. in furnishing electrodes for the work free of charge.

Sediment in Lake Taneycomo

In order to determine the amount of sedimentation that has taken place in the White River (Mo.) area, the Soil Conservation Service of the U. S. Department of Agriculture is conducting a survey of Lake Taneycomo. Soundings will be taken at a number of points in the lake, and the deposit of silt will be penetrated to the original bed, in order to determine the amount of sedimentation that has been washed into the lake since its formation.

Rail Fissures Traced to Shattered Steel and Heavy Wheel Loads

Results of a four-year investigation in laboratory and field prove that the dangerous transverse-fissure type of failure is closely related to shatter-cracking, which may be controlled by careful thermal treatment after rolling

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AN ABSTRACT of Bulletin 376 of the American Railway Engineering Association.

FOR some twenty years the problem of rail failures from internal transverse fissures has been important with respect to both public safety and railway maintenance. Such failures are numerous and in the past have caused many bad train wrecks. Ever since the late James E. Howard first called attention to this type of failure in 1911, transverse fissures have been recognized to be a major railway problem, but in spite of extensive investigation their nature and treatment remained obscure.

Finally a systematic study in laboratory and field under the joint auspices of railways and rail manufacturers was initiated in 1931, and in four years of work this study has cleared up some of the basic questions involved. It has given strong evidence that transverse-fissure failures are closely related to the phenomenon of shatter-cracking in rails and can be controlled by careful thermal treatment after rolling; evidently the shatter-cracking is due to destructive stresses set up during the cooling of the rails on the mill hotbeds. High wheel loads and impact from flat wheels contribute greatly to the danger of fissure formation, it was also found. Improved methods of rail manufacture that are now available give promise of a marked reduction in the development of internal fissures in service.

Cooperative rail investigation

The current rail investigation is under the joint auspices of the Rail Manufacturers' Technical Committee, the Association of American Railroads, and the Engineering Experiment Station of the University of Illinois. The headquarters of the investigation are at the materials testing laboratory of the University of Illinois, and the active test party comprises some eighteen testing engineers, metallurgists, test assistants and mechanicians under the direction of the writer. An advisory committee of eighteen members was appointed by the rail manufacturers' committee and the A.A.R., with F. W. Wood, of Baltimore, and Earl Stimson, chief engineer of maintenance of the Baltimore & Ohio Railroad, as joint chairmen.

The details of the study necessarily evolved during the progress of the investigations. In general, five lines of inquiry were included, though in addition there were many special tests and studies. These five are:

1. A laboratory study of the mechanism of formation of internal fissures in rails under repeated wheel loads.
2. Field studies of the range of magnitude of wheel loads occurring in service.
3. A critical study of acceptance tests for rails and a study of proposed nondestructive tests for rails.
4. Tests of rails made by special processes designed to prevent internal shatter-cracking during manufacture.
5. Service tests of test rails under traffic.

The last-named tests are now in progress. Their results may, after some years of service, modify the conclusions drawn from laboratory tests.

Stress and fatigue in rails

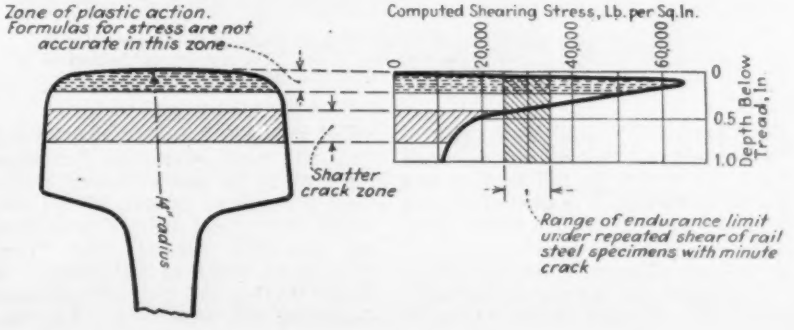
Under traffic, damage to rails may be caused by either of two distinct systems of stress: (*a*) stress due to bending moment; or (*b*) a complex system of stress set up directly under a wheel load. The reports of the joint committee of stresses in railroad tracks of the A.R.E.A. and the A.S.C.E., of which Professor A. N. Talbot is chairman, furnish the necessary mathematical analysis for computing the stresses due to bending moments. (See reports of the joint committee for 1920, 1923, 1925 and 1929). In Bulletin 212 of the Engineering Experiment Station of the University of Illinois, H. R. Thomas and V. A. Hoersch have developed the Hertz analysis for stress in cylinders under radial compression so as to determine the stresses in the metal slightly below the surface of the head of the railroad rail under a wheel load, with special attention to the shearing stresses.

Using the formulas developed in the latter study, Fig. 1 shows the theoretical shearing stress in a rail head due to the direct action of a wheel load. It is to be noted that the steel immediately below the surface is stressed beyond its elastic range, and consequently the formulas do not hold for that region. However, test results indicate a fair agreement of actual with computed stress values at depths of ½ in. or more below the surface, where the nuclei of fissures are usually found.

During the past fifteen years the belief that an internal fissure in a rail is a gradually spreading fracture (in other words, a fatigue failure) has gained ground and may now be considered fully established. The investigations under discussion fully confirm the "fatigue" nature of internal-fissure fractures.

Probably the commonest cause of fatigue failures in structural and ma-

FIG. 1—WHEEL-LOADS set up high shearing stress a short distance below the bearing surface of the rail. (40,000-lb. load on 33-in. wheel; stresses computed by analysis developed by H. R. Thomas and V. A. Hoersch. In the upper zone the steel undergoes plastic deformation, and the computed results here are not accurate.)



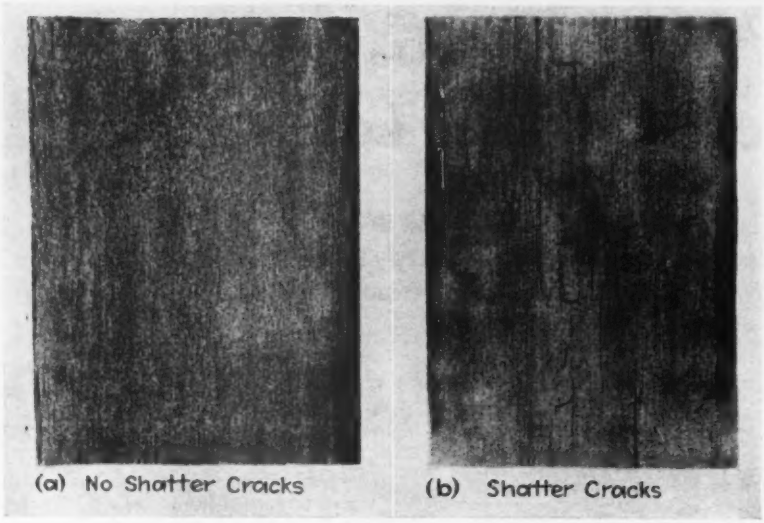


FIG. 2—SOUND AND UNSOUND RAILS as revealed by deep etching of a horizontal slice. Rail at right shows shatter-cracking.

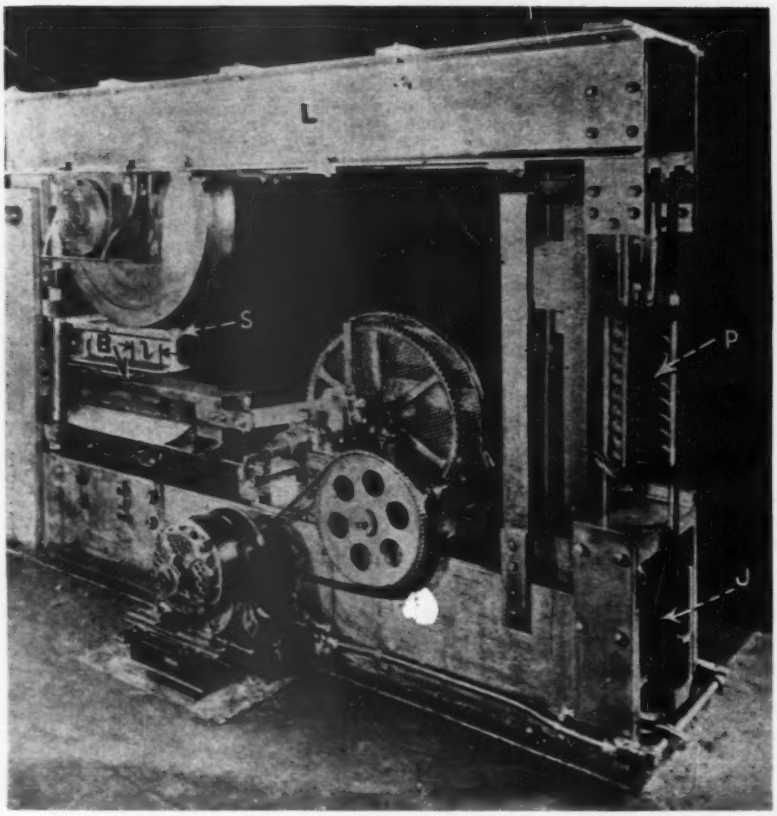


FIG. 3—ARTIFICIAL TRANSVERSE FISSURES were produced by fatigue action in rolling-load testing machines of the kind here shown. The machine and method of operation are described in the adjoining column at the right.

chine parts is localized high stress at "stress raisers," such as notches, small holes and cracks. In rail steel, as was shown by tests made in connection with the present investigation, a minute crack will reduce the strength of the metal to resist repeated stress by about one-half.

For many years the presence of internal cracks (shatter-cracks) in newly rolled rails has been revealed by etch test, and much discussion has taken place as to the cause of such cracks, their presence or absence in rail steel before etching and their effect in starting spreading fissures (See "Deep Etching of Rails and Forgings," by F. M. Waring and K. E. Hofammann, describing etch tests, *Proc.*, Am. Soc. for Testing Materials, 1919, with extended discussion following). The present investigation included extensive work on the presence of such shatter-cracks in new rails and on the effect of these cracks on the strength of the rails.

In these tests on new rails the first step was to cut off a 6-in. test section, about 30 in. from the end of the rail, to slice the head into horizontal strips and to make an etch test. The rail was then tentatively classified as shatter-cracked or not shatter-cracked. The question as to whether shatter-cracks exist in newly rolled rails before etching, or whether they are the result of the etching process, is of only academic interest, because a condition such that a crack can be started by etching or by a slight change in localized stress is about as damaging as is the presence of an actual crack. However, by the use of high magnification cracks were detected in some unetched slices of newly rolled test rail. The difference in appearance of sound and shatter-cracked rails after etching is shown by Fig. 2.

Tests in a rolling-load machine

At the outset of the investigation 165 rails of 130 lb. per yd. and 130 rails of 110 lb. per yd. were furnished for test purposes. These rails came from five different mills and were made in the ordinary way—that is, cooled on hot-beds. The remaining rails in the heats from which these test rails were taken were put in service in railroad tracks at various test locations.

The laboratory study of the way in which internal fissures are formed was carried on by means of special rolling-load testing machines based on the same principle as a testing machine previously used by Prof. W. M. Wilson for tests of bascule bridge rollers. This machine applies cycles of definite and controllable wheel load and bending moment to a specimen length of a rail. Its construction is shown in the accompanying photograph, Fig. 3.

A short specimen of rail, *S*, is moved backward and forward under a wheel load. The load, which can be varied from zero to 80,000 lb., is applied through the lever *L* by means of a screwjack *J* and is measured by the compression of the spring *P*. The maximum bending moment is the product of the load on the wheel and the overhang *l* of the specimen when it is in its extreme left-hand position; the overhang may be varied by moving the block *B*. The stroke of the machine is 7 in., and the speed is 55 r.p.m. A revolution counter is attached to the crankshaft of the machine, and an automatic cutoff switch is operated by the dropping of the lever *L* when the specimen fails.

More than 200 rail specimens have been tested in the three rolling-load testing machines in use, many of them to 1,000,000 or more cycles of stress. Failures due to internal fissures have been developed in a considerable num-

ber (though by no means all) of the specimens cut from newly rolled test rails in which etch tests had shown shatter-cracks. It is to be noticed that by no means do *all* shatter-cracked rails develop fissures either in laboratory tests or in service in the track. No failures starting from internal fissures have been developed in laboratory tests of specimens from newly rolled rails free from shatter-cracking. Whenever such uncracked rails failed in the rolling-load tests, they failed by a fatigue crack that started at the surface of the rail; moreover they required higher wheel loads to cause fracture than did shatter-cracked rail specimens. The two types of spreading fracture are illustrated in Fig. 4.

Confirmatory evidence of the connection between shatter-cracking and transverse fissures was supplied by study of fractured rails received from service. About 77 per cent of such rails showed evidence of shatter-cracks in addition to the fissure that caused the fracture of the rail.

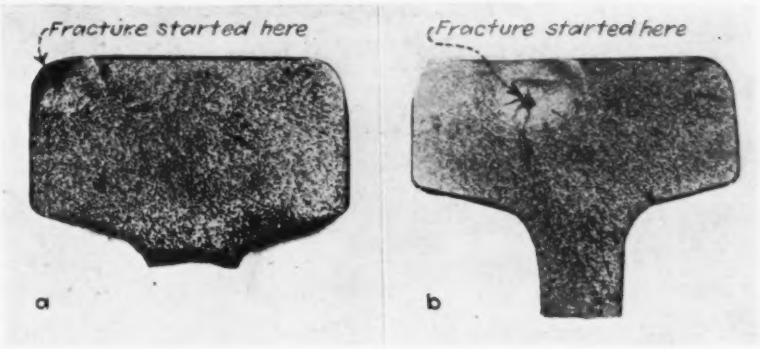


FIG. 4—FATIGUE FAILURE occurred in typically different forms. Rails free from shatter-cracking failed by a fracture starting at the surface. Shatter-cracked rails formed internal transverse fissures at lower wheel-loads than the other rails carried.

Internal fissures and stress under load

In rolling-load tests of rail specimens, internal fissures have been developed at sections where bending moments were very small, but never at sections of heavy bending moment located outside the path of the wheel load. This is evidence that it is the stress due to the direct action of wheel load which *starts* spreading fissures, and a study of the different stresses indicates that the shear stress is most dangerous. A study of rails in which spreading fissures were produced shows, however, that the *direction* in which a fissure spreads is strongly affected by the relative size of wheel load and bending moment. High bending moment tends to cause fissures to spread as transverse fissures, while in rails with low bending moment and high wheel load the fissures tended to spread in a horizontal plane. The direction of a shatter-crack also influences the direction of a spreading fissure.

The stresses directly under a wheel load are practically independent of the size of the rail, being confined to a very small volume of steel. The use of heavy rails would, then, seem to offer no direct relief from the danger of fissures. It may, however, offer some relief in two indirect ways: (1) heavy rail may make smoother riding track and hence reduce the impact stresses set up in rails and wheels; and (2) heavy rail reduces stress due to bending moment, and hence tends to cause any fissure to spread in a horizontal or a vertical-longitudinal plane rather than in a transverse plane.

How the failure of rails in the rolling-load tests is related to shear stress (as computed by the theoretical formulas) is set forth by the test plotting of Fig. 5. The fatigue limit of uncracked rail steel under shear varying from zero to maximum was found to range from 55,000 to 65,000 lb. per sq.in. Fatigue tests of specimens of rail steel with a small crack showed a reduction of fatigue limit to about 50 per cent of that of uncracked steel. The fatigue limit of the steel in the center of the head of a shatter-cracked rail, then, might be expected to range from 27,000 to 32,000 lb. per sq.in. for shearing stress repeated but not reversed—the cycle of stress to which a rail is subjected under the direct action of passing wheel loads.

In Fig. 5 all tests results are for newly rolled rails. It will be noted that all but one of the 24 specimens in which internal fissures developed were subjected to shearing stress within or above the fatigue range for shatter-cracked steel. In the case of those shatter-cracked specimens which failed by fatigue failure starting from the surface, all but six failed under theoretical shearing stress *below* this range, and at high bending stress. Of the 118 specimens that showed no shatter-cracks in the etch test and that failed by fatigue fracture starting from the surface, all but

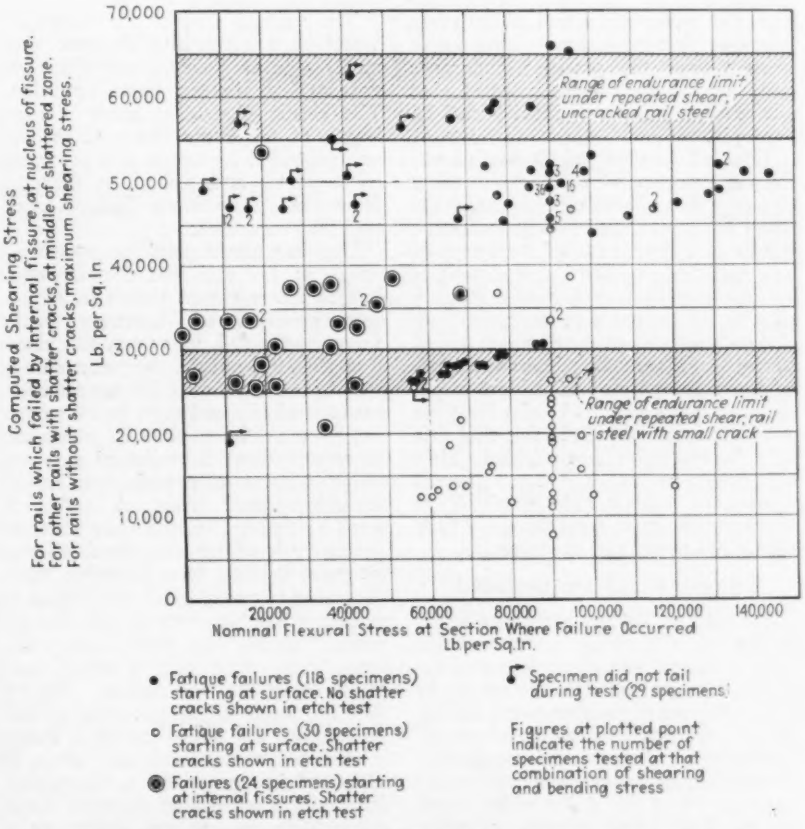
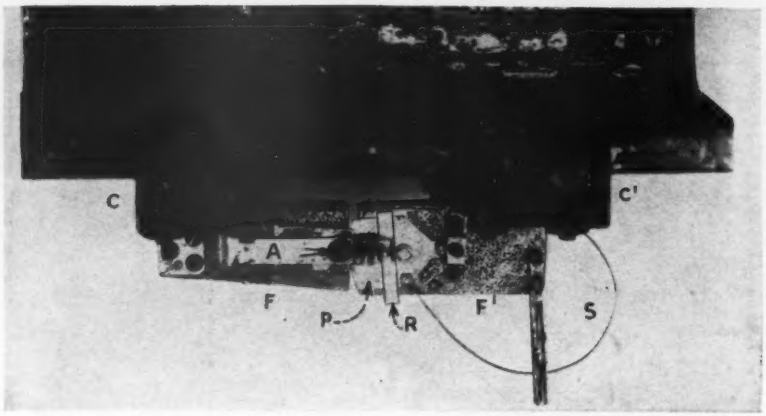
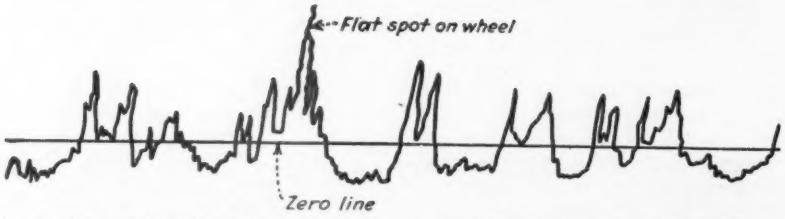

Computed Shearing Stress
For rails which failed by internal fissure, at nucleus of fissure.
For other rails with shatter cracks, at middle of shattered zone.
For rails without shatter cracks, maximum shearing stress.
Lb. per Sq. In.
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
Range of endurance limit under repeated shear, uncracked rail steel
Range of endurance limit under repeated shear, rail steel with small crack
20,000
40,000
60,000
80,000
100,000
120,000
140,000
Nominal Flexural Stress at Section Where Failure Occurred
Lb. per Sq. In.
Fatigue failures (118 specimens) starting at surface. No shatter cracks shown in etch test
Fatigue failures (30 specimens) starting at surface. Shatter cracks shown in etch test
Failures (24 specimens) starting at internal fissures. Shatter cracks shown in etch test
Specimen did not fail during test (29 specimens)
Figures at plotted point indicate the number of specimens tested at that combination of shearing and bending stress

FIG. 5—ROLLING-LOAD TEST RESULTS established that rail fatigue was due to shearing stress directly under wheel load rather than bending stress, and that rails free from shatter-cracking had much higher endurance strength than shatter-cracked rails.





FIGS. 6 AND 7—MEASURING STRAIN in the rail by DeForest scratch extensometer. The instrument is shown and part of a record taken under a passing freight train, stretch magnified 200 times. A flat spot on one wheel produced high strain.

eleven failed at stresses below the fatigue limit for uncracked steel for repeated shearing stress, and all but seven failed at flexural stresses equal to or greater than the probable endurance limit for rail steel under repeated (but not reversed) flexure—about 80,000 lb. per sq.in.

Thus, all the observed phenomena and test measurements—theoretical shearing stress in the railhead directly under the wheel load, results of rolling-load tests, results of fatigue tests of specimens of rail steel, and results of fatigue tests of specimens containing a minute crack—combine to present a consistent picture of the mechanism of development of rail fracture as a spreading internal fissure. At the present time and subject to revision of belief by final results from the test rails in service, it is felt that this rails investigation has yielded fairly convincing evidence that internal fissures in rails usually originate at shatter-cracks that form in some rails during the process of manufacture.

Effective wheel-load measured

The nominal load on any car-wheel is the dead weight which it carries. This latter is known with a fair degree of accuracy. Due to inertia forces set up by lack of balance of rotating parts, closing solid of springs, flat spots and out-of-roundness of wheels, and irregularities of track surface, the actual load under a moving wheel may differ widely from the nominal, being greater in some points along the track and less at others. Increase of wheel load, spoken of as the "dynamic augment," is of importance in considering strength of rail.

The method adopted for measuring wheel load (including dynamic augment) was an indirect one which did not involve altering track conditions. The elastic elongation along a short length of the lower flange of the rail was measured by means of a recording extensometer and, using the formulas developed by Professor Talbot, wheel load was computed.

The instrument used in measuring strain in the rail was the DeForest scratch extensometer, shown in Fig. 6. Two clamps, C and C^1, attached to the flange of the rail, carry two flat pieces F and F^1. One of the flat pieces carries arm A, to the end of which are cemented particles of diamond dust, and the other flat piece carries a plate P, which can be rotated about a horizontal axis and which carries a record strip R of chromium-plated brass. A spring S tends to rotate P and R, but when no stretching is taking place the friction is too great to allow this. However, when stretching is taking place the friction of motion is less than that of rest, and the spring "hitches" the plate a short distance every time that a wheel load passes over the instrument. Fig. 7 shows a magnified record taken by this instrument. The actual stretch of flange is recorded unmagnified, and strain is measured by means of a microscope fitted with a micrometer eyepiece. On a single strip records are drawn by a number of different grains of diamond dust, and usually several of these records are sufficiently clear and legible for measuring.

The experimental determination of the range of wheel loads to be expected in service involves the gathering of a large amount of statistical data. Obviously maximum, minimum and average service values of wheel load must be estimated rather than measured.

Data for preliminary estimates have been obtained from observations at two different test locations: (1) on the B.&O. Railroad north of Dayton, Ohio, and on the Pennsylvania Railroad near Coatesville, Pa. The maximum wheel loads due to freight-car wheels were found to be as high as any loads observed under locomotive wheels, and the results reported here are for freight-car wheel loads. Records of 53,600 wheel loads as recorded at seventeen test locations for instruments were obtained at Dayton, and of 36,800 wheel loads at eight instrument locations were obtained at Coatesville. Fig. 8 shows the relative frequency of loads of different magnitudes.

Both locations represent track in fairly good condition with heavy rail. They represent about average conditions of rolling stock. It is to be noted that at both locations there were occasional wheel loads equal to or greater than 40,000 lb., and it has been noted in rolling-load tests in the laboratory, that spreading internal fissures in shatter-cracked rail specimens were developed by loads as low as 40,000 lb.

At the present time it is not possible to make any quantitative estimate of the relative frequency of high wheel loads caused by track conditions and by rolling-stock conditions. It can be stated with confidence that both are important.

Problem of acceptance tests for rails

The ideal acceptance test for rails would be a non-destructive test that could be applied to each rail. No such test is available at present. Much study and experimentation have been carried on in attempting to find some reliable non-destructive test for shatter-cracks in rails. Tests of change in electrical resistance, of distortion of magnetic field above a crack, of thermal conductivity, of modification of sound waves by cracks—all these have been tried, but no reliable test has been found. The Sperry test is effective for finding fissures but has not proved sensitive enough to detect shatter-cracks in newly rolled rails.

The only acceptance tests for rails that are available today are tests of specimens to destruction. Such tests are better than no tests at all, but they do not give assurance of rails free from shatter-cracks. The standard strength test for rails is a drop test of specimens, usually three, from a heat of steel. Placed head up on supports 3 to 4 ft. apart, a specimen must withstand one blow from a 2,000-lb. weight dropped from a height of about 20 ft.; the

precise height and span depend on the weight of rail under test. Sometimes head-down tests are made. Usually if the specimens withstand one blow, one of them is subjected to repeated blows until it fractures, the number of blows is noted, and the stretch along the lower edge of the rail is measured. This repeated-blow test is not very satisfactory because the first blow so distorts the rail that the stresses set up under successive blows are uncertain and uncontrollable.

The bend test, which has been proposed by a number of engineers and metallurgists, seems to be distinctly better than the drop test. It is a flexural test of a 5- or 6-ft. length of rail made in a testing machine or a hydraulic press fitted with autographic recording equipment for drawing a load-deflection diagram. After fracture, the elongation along the tension edge is measured as in the drop test. The bend test gives all the data of the drop test and load values in addition. The area under the load-deflection diagram measures the energy required for fracture more accurately than does the drop test. Moreover in the bend test the uncertainty of effect of blow due to the distortion of specimen by the first blow is avoided.

The etch test, which consists in etching slices cut from the head of a piece of rail, gives direct evidence of the presence or absence of a shattered condition in the specimen tested by opening up any cracks and making them visible. It also gives some idea whether any cracks present are close to the tread or below the center of the head, where their presence would be less injurious.

Drop, bend and etch tests are all tests of specimens to destruction. Like all tests to destruction these tests leave an element of uncertainty as to whether any specimen tested is representative of worst, average or best conditions of the steel in a heat of rails.

In the course of the present investigation, drop tests, bend tests and etch tests have been carried out on specimens from 74 newly-rolled rails. Table I summarizes the results of drop tests and bend tests as compared with the indications of etch tests. An examination of Table I shows that none of the tests used assure acceptance of all rails free from shatter-cracks, and at the same time rejection of all rails containing shatter-cracks.

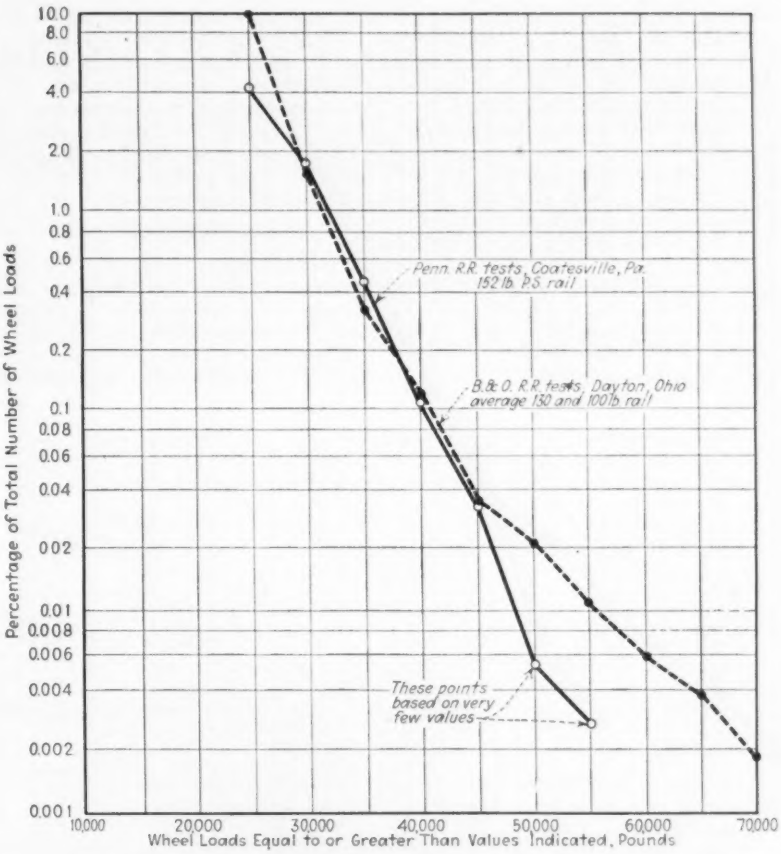


FIG. 8—WHEEL-LOAD FREQUENCY in modern freight service. Full line represents records obtained on the Pennsylvania Railroad near Coatesville, Pa., for 36,800 freight-car wheels. Dotted line represents records obtained on the Baltimore & Ohio Railroad near Dayton, Ohio, for 53,659 freight-car wheels.

Effect of thermal treatment

To gain some idea of the effectiveness of proposed processes for preventing shatter-cracks by the thermal treatment of rails after rolling, tests were made on 36 pairs of rails furnished by various mills. One rail of each pair had been cooled in the usual way on a hotbed, while the other rail, usually from the same ingot as the first and always from the same heat, had been given a special thermal treatment. Two types of thermal treatment were studied: (1) a controlled cooling of the rail after rolling, and (2) a normalizing process in which, after rolling, the rail was allowed to cool through its critical range in air, then reheated above the critical range to a temperature producing grain refinement, and then cooled in air. (See *Railway Age* for March 2, 1935, for description of this process).

From Mill L (two pairs of rails) hotbed rails showed shatter-cracks, low results in bend test and developed fissures in rolling-load test, while control-cooled rails showed no shatter-cracks, good results in bend tests and developed no fissures in rolling-load tests.

From Mill E (five pairs) two hotbed rails showed shatter-cracks, one showed a low result in the bend test, two withstood rolling-load test (2,000,000 repetitions of 75,000-lb wheel load) without fissure; control-cooled rails showed no shatter-cracks, good results in bend test, and under rolling-load test one rail developed failure in web, not a fissure in head.

From Mill D (five pairs) hotbed rails showed many shatter-cracks, gave low results in bend test, and one de-

TABLE I—A STUDY OF RAIL ACCEPTANCE BY VARIOUS TESTS APPLIED TO 74 TEST RAILS

Test	Criterion of Acceptance or Rejection*	Number Rejected: Rails free from Shatter-Crack†	Number Rejected: Rails with Only Longitudinal Shatter-Crack†	Number Accepted: Rails with Transverse Shatter-Cracks†
Bend test				
Energy for fracture	125,000 ft.-lb.	0	2	2
Elongation	12% in 1-in. gage length on tension edge	1	3	2
Drop test, head up, single blow	1 blow, approx. 40,000 ft.-lb.‡	0	0	5
Elongation	7.5% for 130-lb. rail; 8.5% for 110-lb. rail*	3	2	2
Drop test, head down, single blow	1 blow, approx. 40,000 ft.-lb.‡	0	1	3
Elongation	7.5% for 130-lb. rail; 8.5% for 110-lb. rail*	0	1	3

*Arbitrary values, selected by test party.
†Shatter-cracks as shown by etch tests of 6-in. specimen from end of broken rail specimen.
‡A.S.T.M. standard test.

veloped fissure in rolling-load test; normalized rails showed a few shatter-cracks in three rails, gave good bend-test results and developed no fissures in rolling-load tests of three rails.

From Mill B (twelve pairs) no shatter-cracks were found either in hotbed rails or in control-cooled rails. Control-cooled rails showed distinctly higher values in bend tests than hotbed rails. No fissures developed in rolling-load tests.

From Mill I, second lot of Mill E and second lot of Mill D (twelve pairs), no shatter-cracks were found either in hotbed rails or in thermally treated rails, good results were obtained in all bend tests, and no fissures have yet developed in rolling-load tests.

No appreciable lowering of Brinell hardness was observed for any of the thermally treated rails as compared with the corresponding hotbed rails. The tests so far made indicate distinct improvement of quality due to all of the thermal treatments studied.

Low-temperature tests

Rail failures are more frequent in cold weather than in warm weather. Two causes are found for this: (1) changed track conditions due to freezing and thawing; and (2) changed physical properties of rail steel at low temperatures.

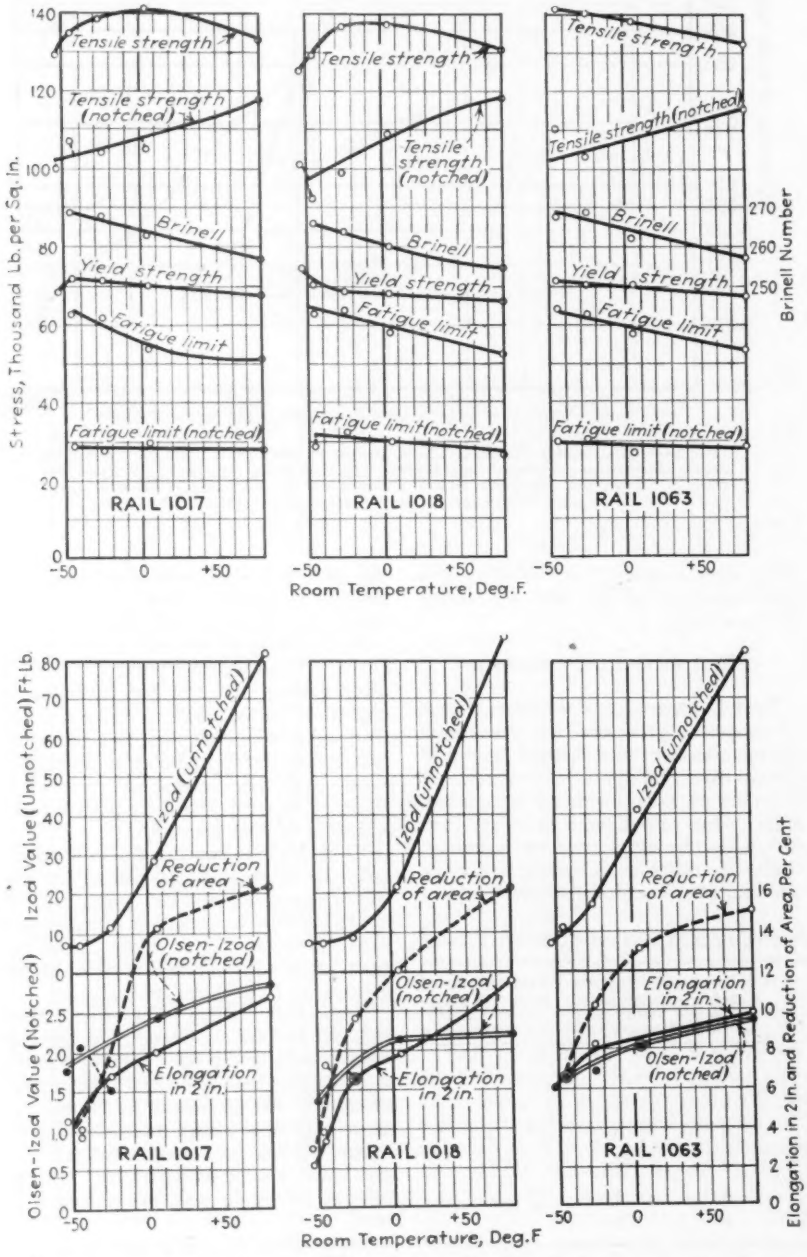


FIG. 9—STEEL AT LOW TEMPERATURE has lessened ductility, according to tests carried down to 50 deg. below zero.

The second of these causes was studied by a series of tests made in the cold room of the U. S. Air Service laboratories at Wright Field, Dayton, Ohio, which were made available for the rails investigation. Tests were made on specimens cut from three newly rolled rails: rail 1017, a hotbed rail in which etch tests had shown shatter-cracks; rail 1018, a control-cooled rail free from shatter-cracks; and rail 1063, a hotbed rail free from shatter-cracks. Fatigue tests, impact tests, tensile tests and Brinell tests were made with minimum temperatures of —40 deg. F. for fatigue tests, and —53 deg. F. for impact tests and tensile tests.

Fig. 9 shows the results of the low-temperature tests. It will be noted that the strength values (yield strength, tensile strength, fatigue limit and Brinell hardness) increase as temperature decreases, with the exception of tensile strength of notched specimens. Ductility values (elongation, reduction of area, impact values both for notched specimens and for unnotched decrease, as found in previous tests elsewhere.

Metallographic studies

In connection with the work of the investigation, many metallographic studies of the structure of rail steel have been carried out and also studies of the effect of nitrogen and oxygen in rail steel. These studies have failed to show any correlation with the presence or absence of a shattered steel condition.

Studies of the internal strains in rail-heads have shown rather small strains, and in the shatter-crack zone the strains have been found to be compressive rather than tensile, checking results found by other investigators. It should be noted that the strains left *after cooling* give no measure of the strains set up *while the rail is cooling.*

Comparative tests of specimens from near the surface of rails with tests of specimens from the center of the head have been made on a few rails. Specimens from rails in which etch tests had shown shatter-cracks gave distinctly lower values of tensile strength, fatigue strength, impact values, elongation and reduction of area than did specimens from rails in which etch tests had shown no shatter-cracks.

This report is a progress report, and the behavior in service of test rails may throw new light on the problem of internal fissures. The results of the investigation so far emphasize the importance of freedom from shatter-cracks in rail steel; indicate that thermal treatment of rails after rolling may markedly reduce trouble from this source; emphasize the importance of careful track maintenance and of maintenance of rolling stock in reducing the actual wheel loads to which rail is subjected; and emphasize the desirability of developing a more effective acceptance test for rails than the drop test now in common use.

The Construction Code and What It Taught

[*In Two Parts*]

PART I—After months of effort and controversy had finally led to agreement on an NRA code for the complex construction industry, many difficulties were encountered in administering the code, and full success in administration was not yet in sight when the NRA system was ended

BY JAMES W. FOLLIN

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WHEN the Construction Code Authority was dissolved last July, shortly after the United States Supreme Court had decided that the NRA code system of regulating industry was unconstitutional, two years of labor to coordinate and stabilize the operations of construction came to naught. The influence of this event on the future of the complex activities embraced in construction remains to be appraised.

In the meantime, however, certain questions demand present answer. What did the administration of the code accomplish for the advancement of construction? Do the purposes of the code appear worthwhile in retrospect, and to what extent were they attained? Did experience with the code show construction self-government to be a practical possibility? As a partial answer to these questions the history of code administrration is here outlined.

It has long been recognized by thoughtful leaders of construction that its future stability and progress depend on integration of its many diverse and discordant units into a coordinated industrial system. With this in mind, let it be said at the outset that the construction code fell far short of achieving this integration. It proved to be unable to control all the recalcitrant elements of the industry, probably in large part because its objectives were not fully understood.

The regrettable fact is that, although the two years of code-making and code administration abounded in examples of unselfish cooperation, the industry as a whole had not grasped the significance of what was being attempted. If such was the result of self-regulation conducted under governmental auspices and enforceable by law, it is a grave question what can be expected from voluntary action alone.

A—Drafting the Code

UPON formation of the National Recovery Administration in June, 1933, the decision to join together all construction codes under a master code was reached by the industry quickly, it will be recalled. Even before the National Industrial Recovery Act had received its finishing touches, several construction groups had started to develop codes of fair competition, most actively the general contractors, the structural-steel fabricating group and the electrical contractors. Codes for two of these groups were early submitted to the NRA. Confronted by such facts, leaders of the Construction League as well as NRA officials recognized the necessity of tying together the numerous elements of construction through a master or basic construction code, to which would be attached supplemental codes for the several distinct groups. The league voted on July 26 to undertake the sponsorship of such basic code, and its committee began operations the same week.

An advisory council brought together by this committee, comprising representatives of the various national trade associations concerned, met on July 31, 1933, and discussed, revised and finally unanimously adopted a draft of the basic code. This draft was submitted to NRA on the afternoon of Aug. 1.

Organized labor expressed immediate disapproval; it objected to the wage and hour provisions and to the make-up of the Code Authority, which did not include labor representation. In spite of this opposition, however, the wage and hour provisions were accepted two days later for inclusion in the President's Re-employment Agreement when signed by construction employers. The basic code and as many supplemental chapters as were ready were slated for public hearing on Sept. 5.

Dissension in the industry

The hearing lasted three days, while arguments pro and con on the basic code and nine of its supplemental codes were presented. Labor brought in an extensive analysis of wage and hour conditions in the construction industry in support of the 30-hour week as opposed to the 40-hour week proposed in the code, and in support of minimum wages for unskilled and skilled employees conforming to the PWA regulations—i.e., 40, 45 and 50c. per hour for unskilled labor, and $1, $1.10 and $1.20 for skilled labor, in the southern, central and northern zone respectively. The proposed basic code set a minimum for unskilled labor of 40c. per hour, with reduction to 30c. per hour under certain conditions (unfortunately this reduction was later abandoned) and fixed no minimum rates for skilled workers. Various of the supplemental codes heard at the public hearing, however, proposed minimum rates for skilled workers either conforming to PWA regulations or based on an ascending scale according to the population of the trade area and starting, in some cases, as low as 60c. per hour.

Assimilation of the lengthy testimony and preparation of necessary supporting statistics occupied several weeks following. About the time the basic code was ready to be submitted for final approval, opposition appeared within the industry in the form of a demand for a code for building work separate from that for open construction and highway work. The movement was sponsored principally by the National Association of Building Trades Employers, with support of many of the builders' exchanges and organized labor. Approval of the basic code, accordingly, was deferred until a second public hearing was held on Nov. 20, 1933, which was to test the representation of the groups sponsoring the basic code.

The demonstration of industry support at this second hearing was so convincing that organized labor withdrew its opposition to the basic code, but not to the labor provisions. Accordingly, NRA Administrator Hugh S. Johnson appointed a conference committee comprising representatives of the sponsors of the basic code and supplemental codes, of organized labor, and the groups which had sponsored the separate building code.

The code is approved

Chief Counsel Donald R. Richberg was the leading figure in the meetings of this conference group. Labor demanded clarification and improvement of the procedure for area agreements or collective-bargaining agreements under Section 7 (b) of the act; this was granted by adopting language substantially as approved in the final draft. Labor also demanded a 30-hour week, higher minimum-wage rate for unskilled workers, and inclusion of minimum rates for skilled workers. These latter demands were not agreed to. The basic code and nine supplemental chapters went to the President for final approval on Dec. 22, 1933.

Labor's opposition, however, prevailed in that the President did not sign the code but sent it back to iron out the differences. Major George L. Berry, division administrator in NRA, thereupon appointed a conference committee representing employers and labor and proposed the establishment of an industrial relations board in the code composed equally of employers and employees—the National Construction Planning and Adjustment Board. The creation of this board, which was the most forward step taken in any code up to that time in the industrial relations field, was the master stroke that resulted in labor's withdrawal of opposition to the code.

Code Authority organizes

Once the President had signed the basic code in this form—it was now called Chapter I or General Provisions —on Jan. 31, 1934, the organization meeting of the Construction Code Authority (CCA) was called for Feb. 15, to prepare for administration of the code, which became effective on March 2. While the six months spent in code-making seemed at the time to be unduly long, now the wonder is expressed that a constitution for the country's second largest industry, considered the best NRA code, could have been completed so satisfactorily in so brief a time.

The constituency of the CCA had been modified in the Richberg conferences. Instead of a group of nine appointed by the Construction League and balanced to represent the several fundamental elements of the industry, there was set up a body comprising a member named by each sponsor of the code, except that the general contractors were given four members because of the subdividing of their divisional chapter into three parts: builders, heavy construction, highway and public works. The sponsors listed in the code initially numbered 20, which gave a voting membership of 23, plus three administration members without vote. Provision was made for additional members when new chapters should be added. When the Code Authority dissolved, there were 29 voting members, while two members appointed by additional chapters had not yet been seated.

CHAPTER	APPROVED	EFFECTIVE
I—General provisions	Jan. 31, 1934	Mar. 2, 1934
II—General contr.	Feb. 17, 1934	Mar. 19, 1934
IIB—Heavy construction and railroad contr. subdiv. of gen. contr.	Apr. 29, 1935	May 9, 1935
IIC—Highway contr. subdiv. of gen. contr.	Mar. 16, 1935	Mar. 26, 1935
III—Painting, paperhanging and decorating	Mar. 12, 1934	Mar. 22, 1934
IV—Elev. manufacturing	Mar. 21, 1934	Mar. 31, 1934
V—Cement-gun contr. (later air-applied concr.)	Mar. 21, 1934	Apr. 4, 1934
VI—Electrical con.	Apr. 19, 1934	Apr. 30, 1934
VII—Roofing and sheet-metal contr.	May 10, 1934	May 25, 1934
VIII—Mason contr.	Apr. 19, 1934	Apr. 30, 1934
IX—Tile contr.	Apr. 2, 1934	Apr. 12, 1934
X—Plumbing	May 15, 1934	June 4, 1934
XI—Kalamein	June 9, 1934	June 19, 1934
XII—Wood-floor contr.	May 29, 1934	June 28, 1934
XIII—Resilient-flooring	May 29, 1934	June 13, 1934
XIV—Insul. contr.	June 8, 1934	June 18, 1934
XV—Terrazzo and mosaic contr.	July 13, 1934	July 23, 1934
XVI—Heating, piping and air-conditioning contr.	July 25, 1934	Aug. 9, 1934
XVII—Marble contr.	Aug. 11, 1934	Aug. 21, 1934
XX—Plastering and lathing	June 27, 1934	July 27, 1934
XXI—Stone-setting	Dec. 31, 1934	Jan. 30, 1935
XXII—Building gran.	Aug. 20, 1934	Sept. 19, 1934
XXIII—Construction news service	Dec. 20, 1934	Dec. 30, 1934
XXIV—Cork insulation contr.	Apr. 1, 1935	Apr. 11, 1935

The CCA organized with Stephen F. Voorhees as chairman, John P. Hogan vice-chairman, F. P. Byington treasurer, and the writer as secretary. Several months later B. R. Value, a deputy administrator of NRA, was chosen executive director. The Code Authority was incorporated under the laws of Delaware and established its principal offices in the National Press Building, Washington, D. C.

At once the CCA was deluged by requests from local organizations, particularly from builders' exchanges, for appointment as administrative agencies for the code. However, Chapter I of the code conferred administrative power not on the CCA but on the divisional code authorities provided for under each additional chapter. Accordingly, administration of the code, even of the general provisions of Chapter I, was the function of NRA until approval of the respective divisional chapters and the establishment of administration thereunder were confirmed. This process was necessarily slow, and as late as six months after Chapter I had been approved certain chapters were still awaiting approval; the highway contractors' subdivision of the general contractors' chapter was not approved until March 16, 1935, the heavy construction and railroad subdivision not until April 29, 1935, while the building contractors' subdivision was never approved and was left stranded when the decision of May 27, 1935, killed off the code system.

Moreover, at the end the chapters for architects and professional engineers seemed to be farther from approval than ever. These chapters had been pending from the very beginning and were counted upon to complete the integration of the design elements with the contracting elements. Representatives of the architects and engineers were sitting on the Code Authority, and, as officers, were directing its administration. While approval of the presentation of these design chapters had been given by the respective professional societies, it was no secret that many architects and engineers disapproved of such action.

So far as the major contracting chapters are concerned, the complete code structure was not approved until late in the year 1934, and in view of the delay in establishing administration, the complete structure may be said never to have been entirely in operation. Thus, the code never had an actual trial as an integrated document. However, several of the more important divisional chapters, such as those for general contractors and for electrical contractors, had been in administrative operation for about a year when the end came.

B—Administrative Experiences

THE CODE unfortunately was not complete, for it lacked at least two essential groups. One of these groups, the river and harbor improvement contractors, obtained approval of a separate code, though later its operations were coordinated with those of the construction code through an inter-code agreement. The structural-steel fabricators and erectors applied for a separate code, but in July, 1934, they were granted a code under an administrative order which, aside from other changes, placed the erecting functions under the construction code (*ENR*, July 19, 1934, p. 94). As the fabricators refused to accept such modified code, it was suspended, and in the end this branch of the industry had no code except as its erecting operations might be considered to have been governed by the construction code.

The speculative builders (land development and home-building group), sponsored by the National Association of Real Estate Boards, attempted through-

out the entire NRA period to obtain a separate code for its operations. Its efforts failed, but in the meantime the speculative builders were exempted from the construction code except as to wage and hour provisions. They paid no code asssessments and made no contribution to the progress of the code.

Problems of local integration

First and most far-reaching of the weaknesses of the system was that the code brought about only a highly imperfect integration of the construction industry. The integration covered only the national phase; the divisional groups were left with almost complete autonomy to administer the general provisions and the provisions of their specific chapter among their members, without coordinating such efforts with other divisions.

Each divisional code authority was authorized to establish local administrative agencies and to delegate to them administrative power. When such administrative power was communicated separately by some twenty separate divisional code authorities, each single locality was governed by a multiplicity of code agencies. Integration, to be effective, necessarily must reach into the locality where the construction work is done; otherwise, separate divisional agencies administering the provisions of separate chapters almost surely would take action contrary to the interests of the members of other divisons. This would destroy integration and cause confusion and conflict.

A case in point was the ruling of certain divisional administrative agencies that sub-bids by their members must be withdrawn when the code provisions as to filing duplicate copies of bids or selling below cost had been violated; this ruling left the general contractor high and dry and unable to accomplish the job at the price he bid, which was based on the withdrawn sub-bids. The NRA finally overturned this ruling. It held that an administrative body could not set up penalties of its own, such as demanding the withdrawal of a bid, but must rely upon reporting violations of the code to the compliance division of NRA for prosecution. However, it was found to be impossible to control the actions of many administrative committees without setting up local machinery to integrate and coordinate code activities.

Local code council plan

Entirely aside from the requirements of effective administration, the excessive cost of supporting a multiplicity of separate agencies became another reason for local coordination. Officials of NRA therefore called upon the Construction Code Authority in August, 1934, to study the possibilities of consolidating field agencies. In its study the CCA, with the help of Prof. Wm. S. Carpenter and H. L. Childs, of Princeton University, experts in governmental procedure, evolved a plan for local code councils, consisting of representatives of all divisions administering the code in the locality and providing balanced voting as between design, general contracting and special or subcontracting elements.

The local code council would take over administration of divisional chapters in their entirety where desired; elsewhere it would coordinate activities and set up trade-practice panels for hearing complaints, adjusting differences and making recommendations for enforcement. The council was also to operate a joint bid depository covering all members of the industry.

Unfortunately, this interesting proposal when submitted to the divisional code authorities received only lukewarm support, partly on account of craft jealousies but largely because of hesitancy in surrendering divisional autonomy. From similar motives a proposal to modify Chapter I, to permit the CCA to establish joint bid depositories where desirable (but continuing divisional bid depositories in operation until such official depositories were established), was tabled by the CCA itself at a meeting in April, 1935.

However, it seemed clear that, had the construction code been re-enacted under an extended NIRA, the National Recovery Administration would have insisted upon such local integration; its approval of divisional budgets would have provided the necessary leverage. It has been stated unofficially that NRA intended to enlarge the powers of the Construction Code Authority. Such action, despite its appearance of centralization, would in fact have resulted in decentralization, since the responsibility for code administration would have been delegated more completley to local integrated industry groups, and code assessments would largely have remained in local hands to support effective administration. Recognition of the separate divisional elements in the industry, however, would have been continued through their inclusion both in the local code councils and in national committees, which would safeguard the interests of the members of such crafts.

Even without the acceptance of an industry plan for local cooperation and integration, individual communities had gone ahead on their own initiative to establish joint bid depositories and otherwise to coordinate code administration. Outstanding was the San Francisco effort, established by a temporary co-ordinating committee appointed by the CCA in the spring of 1934. Seven divisions of the industry were using the joint bid depositories; the territory covered was, in most cases, the northern half of California. The central depository also established subsidiary depositories in outlying towns. In Cleveland nine divisions of the industry were using a central bid depository. In Wilmington, Del., Birmingham, Ala., and Louisville, Ky., joint bid depositories with an appreciable number of divisions participating were in operation (*ENR*, May 15, p. 711). The coordinating committees which brought about these results were among those appointed by the CCA in each of the 53 NRA compliance districts.

Financial difficulties serious

Undoubtedly the most serious problem confronting code administration was adequate financing.

The divisional code authorities developed their own plans of administration and secured approval of their budgets by NRA before code assessments were made. On the whole, they were unable to raise adequate funds. Chief among the reasons for this was that NRA would not prosecute members of the industry for failure to pay code assessments—largely, it is believed, because of doubt as to whether the courts might consider compulsory assessments as taxation that could not be delegated by Congress. Fortunately, much major construction work was financed by governmental outlays, and through the cooperation of government officials the contractor was generally bound to pay the code assessment.

Budgets as approved for the 22 divisions of the construction industry totaled more than $8,000,000, but collections thereon were at the rate of barely $1,500,000 a year. The general contractors' division was in many respects the most efficient in making code collections; of a budget of $2,700,000 a year it collected about $1,000,000.

Some divisions had difficulty because of the poor methods under which they levied assessments. Several others had difficulty because of Administrative Order X-36, which exempted a member of an industry from contributing to other than his principal-line-of-business code authority; when members who operated under other codes as well as construction did not pay, other members also were disinclined to pay. The most acceptable method of assessment was found to be charging for registration of contracts, and the poorest to levy a charge proportioned to volume of business done in some past period.

Collections and expenditures of the Construction Code Authority reached only about 40 per cent of its budgeted $290,000 a year. Its revenue was derived principally from registration fees on projects of $2,000 or over, supplemented later by a flat assessment on the receipts of divisional code authorities.

Another type of financial difficulty was illustrated by a controversy between general contractors and masonry contractors over the question of whether general contractors should pay assessments to the mason contractors' code authority on masonry work performed by the general contractors' own forces. This was resolved in favor of the general con-

tractors, on the ground that the sponsors of the mason contractors' chapter represented only masonry contractors who specialized in masonry work and usually performed it for general contractors.

Such experience demonstrated that there would have to be a coordination of assessments, with collection through such bodies as the local code councils. The alternative would be for the government to make and distribute the collections, or else to support code administration from government funds.

Appeals board not utilized

When the initial proposal for a small code authority (nine members, with voting power balanced between the several functional elements of the industry) was changed during the deliberations of the Richberg committee to provide a code authority of large membership, in which the special or subcontracting elements greatly predominated, provision was made in the code for a Construction Appeals Board. The board was to include an architect, a professional engineer, three general contractors and four special contractors, and was empowered (*a*) to decide conflicts between the provisions of different chapters of the code, and (*b*) to determine appeals from any decision or order by a divisional code authority, primarily in trade-practice complaints.

This board established rules for hearing appeal cases in the spring of 1934, but thereafter was never called into action. Earlier the Construction Code Authority through a committee had heard two trade-practice complaints; in the latter months of the code, trade-practice appeals were taken directly to the NRA. Thus the appeals board, while designed to give the industry maximum self-government, did not succeed in this respect. The fact is, it interposed another step in the already-too-long procedure of adjusting complaints, from the initial handling by a local committee to final determination by NRA, which alone had power to initiate prosecution.

The controversy between the general contractors and the mason contractors referred to above was a typical instance of conflict between provisions of two chapters that might have been determined by such an appeals board. However, in that instance, neither party to the controversy appealed to the board, and the board had no power to take jurisdiction.

In the second and concluding section of this article, to appear next week, the author discusses the difficulties arising from conflicts of the construction code with other codes, from labor and wage questions, from lack of enforcement and other sources of trouble. Then, after a brief summary of experience with the code, he sets forth the chief gains resulting from the code and indicates future possibilities of industry regulation.—EDITOR.

Federal Legislation Affecting the Construction Industry

Summary of the bills passed at the recent session of Congress that will affect the construction industry—Two important bills promoted but not passed

SEVERAL BILLS that already have had bearing upon the construction industry or that will be of importance to it when they become effective were passed by Congress during the session closed last month. Summaries of them are given below, together with summaries of two bills that were actively considered but not passed.

Labor Disputes—This law creates the National Labor Relations Board, to be composed of three members receiving salaries of $10,000 per year each. The board is to investigate charges of unfair labor practices by employers and is empowered to issue cease and desist orders, with the duty of petitioning for enforcement in any circuit court of appeals. Cases dismissed also are subject to review in the federal courts.

Relief—Appropriated $4,000,000,000 to be available until June 30, 1937. Limits of expenditure are set up as follows: highways, routes, streets, and grade-crossing elimination, $800,000,000; rural rehabilitation, relief in stricken agricultural areas, water conservation, trans-mountain water diversion, irrigation and reclamation $500,000,000; rural electrification $100,000,000; housing $450,000,000; assistance for educational, professional and clerical persons $300,000,000; Civilian Conservation Corps $600,000,000; loans or grants, or both, for projects in states, territories and possessions, and self-liquidating projects of public bodies $900,000,000; sanitation, prevention of soil erosion, prevention of stream pollution, seacoast erosion, reforestation, forestation, flood control, rivers and harbors and miscellaneous $350,000,000. The President has authority to increase the limits by 20 per cent should it become advisable to do so.

Social Security—This law provides federal aid to the states for unemployment compensation, old-age assistance, old-age benefits, assistance for dependent children, maternal and child health services, public health work, and aid to the blind. Unemployment compensation is to be financed by taxes based on payroll, to be 1 per cent in 1936, 2 per cent in 1937 and 3 per cent in subsequent years. Employes will pay an income tax of 1 per cent of the wages received in 1937, 1938 and 1939; of 1½ per cent in 1940, 1941 and 1942; of 2 per cent in 1943, 1944 and 1945; of 2½ per cent in 1946, 1947 and 1948; and of 3 per cent in subsequent years. For old-age assistance the federal government matches amounts that may be paid by the state up to $15 per month. Old-age benefits, to begin in 1942, are payable to those who have reached the age of 65, regardless of the need of the recipient. An annual appropriation of $8,000,000 is to be made for the public health work, to be divided on the basis of population, special health problems and financial need of the respective states. The act is to be administered by the Social Security Board.

RFC Extension—This law authorizes the RFC to continue its operations until Feb. 1, 1937. It liberalizes provisions for railroad and industrial borrowing, authorizes the corporation to make loans on stock of national mortgage associations, increases the limit set on purchases of preferred stock in insurance companies, assigns additional duties to the Export-Import Bank, makes provision for loans to mining companies and to those seeking to develop mining properties.

Extension Emergency Railroad Transportation Act——Carriers are required to continue the payment of $2 per mile per year for the support of the office of the Coordinator of Transportation.

Additional Home-Mortgage Relief—This act liberalizes the Federal Home Loan Bank Act, so as to allow home mortgages to be carried over longer periods and under easier conditions. It is intended to go further than relieve home owners in distress, in that it encourages home-mortgage lending and facilitates the insurance of accounts in home-financing institutions. The bond issue authority of the Home Owners Loan Corporation is increased to $4,750,000,000. The original act authorized bonds to the extent of $2,000,000,000. This subsequently was increased to $3,000,000,000, but since applications were received from 1,700,000 home owners it was found necessary to increase the corporation's resources. The act extends the limits for the insurance of mortgage loans so as to permit the insurance of loans on industrial and large properties to an amount not exceeding $50,000. The required capital of national mortgage associations was reduced from $5,000,000 to $2,000,000, to encourage the formation of these associations.

Soil Erosion — This act authorizes surveys, investigations and research as to methods of erosion control. The act points out that control of erosion is essential to conserve water, to control floods, to prevent soil wastage, to prevent the silting of reservoirs, to the maintenance of navigation and to protect public lands. Cooperative arrangements with land owners are authorized.

Farm Research—This law authorizes the Secretary of Agriculture to conduct research into the broadest aspects of the laws and principles underlying agriculture, including research into conservation and use of land and water resources for agricultural purposes. For this work an appropriation of $1,000,000 per year is authorized for a period of five years. At the end of the five-year period the appropriation for research is to be increased to $5,000,000 per year. This is to be a special research fund, to be used in addition to other research appropriations.

Rivers and Harbors—Contains 253 projects, of which 79 projects already were under construction through the use of public-works funds. For that purpose $90,951,199 had been allocated. Projects in the bill which have not received PWA allotments are expected to cost $186,862,315. This is the first rivers and harbors bill enacted since July 3, 1930. Provision is made in this bill for increasing depths in eighteen of the principal lake ports to 24 ft. Provision also is made for the deepeninng of the New York Barge Canal. Projects carried in the bill which are expected to cost in excess of $1,000,000 follow: Columbia River, Wash., $43,900,000; Beaver and Mahoning Rivers, $37,000,000; Upper Mississippi, $32,600,000; Cape Code Canal, $31,838,000; New York and New Jersey channels, $27,000,000; Chesapeake and Delaware Canal, $12,600,000; Sacramento River, $12,390,000; Kanawha and Ohio Rivers, $12,200,000; Boston Harbor, $4,800,000; Grey's Harbor, $4,565,000; Delaware River above Philadelphia, $3,800,000; New York Harbor, $3,700,000; Tampa Harbor, $3,675,000; Miami Harbor, $3,300,000; Allegheny River, $3,000,000; Illinois Waterway, $2,800,000; Keweenaw Waterway, Mich., $2,600,000; Houston ship channel, $2,500,000; Sabine-Neeches Waterway, $2,260,000; San Diego Harbor, $1,800,000; waterway from Choctawhatchee Bay to West Bay, Fla., $1,700,000; Savannah Harbor, $1,400,000; Sturgeon Bay, Lake Michigan, $1,300,000; Port Everglades, $1,100,000; Black River channel, $1,000,000.

Disaster Loan — This act extends through 1936 the authority of the RFC to make loans to non-profit corporations for the repair of damages caused by flood or other catastrophies, originally limited to the year 1933.

The two following bills were considered by Congress but did not pass; the Walsh bill, setting up special limitations on contractors doing government work, was actively opposed by the construction industry.

Flood Control — An omnibus bill, carrying some $500,000,000 in projects, was passed by the House but stopped in the closing days of the session in the Senate by a filibuster. The measure was returned to the Commerce Committee. Majority leaders in the Senate say it will pass at the next session. Bills that would have authorized the modification of the flood-control plan for the Mississippi River did not emerge from the House Committee, due to local differences and to the fact that members of the committee with projects on other streams declined to report out non-controverted portions of the Markham plan.

Contract Relations—The Walsh bill was intended to give the President power to determine minimum wage, maximum hour and minimum age standards to be incorporated in all contracts between private industry and the government, as well as contracts in which the government participated by loaning or granting funds to local governments or private borrowers. Presumably such standards would be equivalent to NRA codes, adjusted as might be necessary.

Provision was also made for the application of the standards to existing contracts which are sufficiently flexible so to permit, and to legalize bids made upon the basis of code compliance, in which case the new standards would be substituted for the old codes.

Steel-Cylinder Concrete Pipe Cast in Long Sections

CONSTRUCTION of the Green River gravity water-supply line in Tacoma, Wash., required the installation of 34,000 ft. of 54-in. lock-joint steel-cylinder concrete pipe, all of which was precast in 16-ft. lengths. The units, weighing 9¼ tons per section, were rolled from a loading well in the storage yard to six-wheel trucks, which hauled them an average distance of 12 miles to the job.

Designed to carry a hydraulic head as high as 240 ft., each section was made up of a 14-gage welded-steel cylinder, with special joint rings attached to each end. This assembly was tested under hydrostatic pressure, so that all welded parts could be observed for possible leakage.

Two cages of reinforcing steel, weighing 80 lb. per foot of pipe, were placed around the steel cylinder, and the complete assembly was placed in the pipe mold. A locomotive crane, operating through the center of the pouring yard, set up the sections and conveyed concrete in a 1¼-yd. bucket to a pouring platform on the top of each mold. Vibration machines and tamping devices on these molds were operated simultaneously with each pour. After being steam-cured overnight, the pipe was removed from the forms, and steam-curing was continued for another 24 hours before the pipes were moved into the storage yard. Here they were kept wet for seven days and were not disturbed thereafter until fourteen days had elapsed from the date of pouring. Pipe was made at an average rate of twenty sections per day.

The contract for the manufacture and installation of the pipe was executed by the American Concrete Pipe Co., of Washington. For the city of Tacoma, the project is under the direction of Ira Davisson, commissioner of public utilities, and W. A. Kunigk, superintendent of the water department.

POURING PLATFORM in plant where 54-in.-diameter steel-cylinder concrete pipe for the Tacoma water supply is being manufactured in 16-ft. lengths.

Flexural Resistance of Shallow Concrete Beams

Large increase in load-carrying capacity results from increased steel percentage—Tests indicate justification of working stresses much higher than present standards

By Conde B. McCullough
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TO DETERMINE the ultimate strength of shallow reinforced-concrete beams or slabs in flexure and in shear, the bridge division of the Oregon highway department recently made a series of tests to destruction on full-size specimen beams of four types, cast under conditions comparable to construction in the field. The four types differed only in the manner of web reinforcement; five beams of each type were tested, each with different percentages of reinforcement. Results emphasize the excessively wide margin between actual ultimate loads and loads permitted by present standards. If the quantity of steel is increased from 1 to 2 per cent, it appears that the ultimate load is practically doubled. Further, with reinforcement percentages commonly used in slabs, failure results from tension in the steel. Hence no significance attaches to unit stresses specified for the concrete and, aside from economy considerations, the way to more effective design is by increased steel percentage.

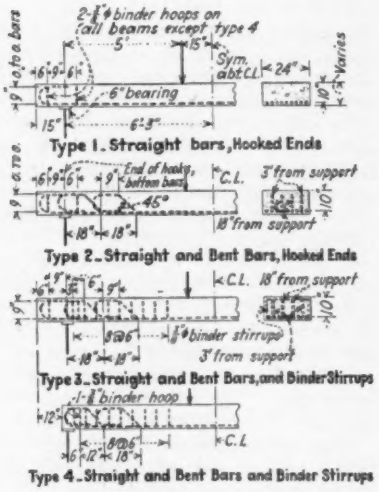


FIG. 1—BEAMS with various arrangements of steel that were used as test specimens.

The purpose of these tests was:

1. To determine the effect on flexural capacity of a variation in the percentage of reinforcement, with special reference to possibilities in heavily reinforced shallow-beam construction for limited clearance conditions.

2. To investigate the effect of web reinforcement in such shallow concrete beams or slabs.

3. To check the suitability of the widely divergent specification requirements for unit working stress in flexure in common current use.

The beams selected for the first test group are indicated in Fig. 1, and include twenty specimens 24 in. wide, 12½ ft. between supports and 10 in. in effective depth. This test group was divided into four sub-groups, differing principally in the manner of web reinforcement. In all cases hooked ends were used for tension reinforcement, and the concrete adjacent to supports was confined against splitting and spalling by means of two ⅜-in. round binder hoops.

Beam specimens

As indicated, type 1 beams were cast with no web reinforcement whatsoever; type 2 beams were web-reinforced with bent bars only; while type 3 beams employed a system of binder stirrups in addition to the bent-up bars. Type 4 beams were identical with type 3 as regards web reinforcement but were cast 1 ft. shorter at each end, to limit terminal reinforcement anchorage to 3 in.

For the individual beams in each of the above type groups the percentage of reinforcement was varied from 1 to 4.6 per cent, this increase being, in general, effected by increasing the bar diameters while keeping the number of bars constant. Data regarding the reinforcement of each of the beams in the various groups are as follows:

Number and Size of Bars	Per Cent of Reinforcement
8—¾ in. round	1.02
9—¾ in. round	2.00
9—1 in. round	2.95
9—1¼ in. round	3.73
9—1½ in. round	4.60

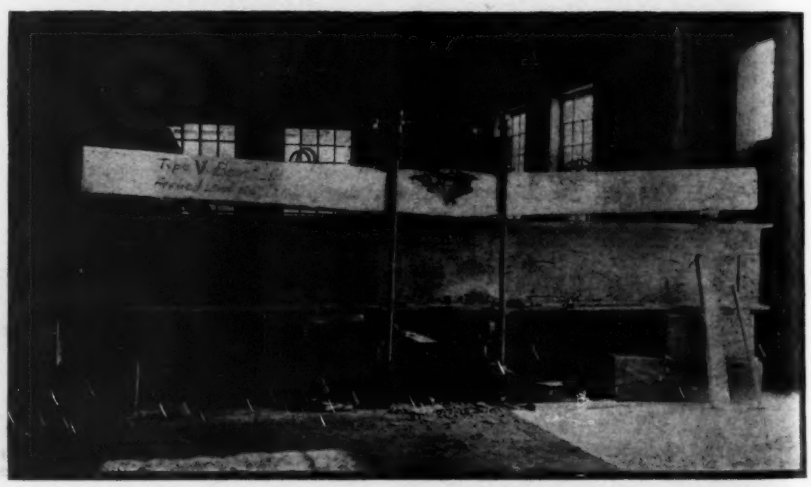
The concrete employed was designed for a 3,000-lb. cylinder strength at 28 days, although the test cylinders actually developed a somewhat greater strength. Throughout the operation of casting, attempt was made to avoid laboratory refinement and to reproduce as nearly as possible such a concrete as might be expected under average field conditions. Aggregates were measured roughly by volume, the material being taken from small open-air stockpiles. A 7-cu.ft. machine mixer was used. The specimens were cured under moist burlap until tested at the age of 28 days. The reinforcement employed was of intermediate grade, with a yield point ranging from 45,000 to 55,000 lb. per sq.in.

The loading set-up is indicated in Fig. 2. The specimens were placed on bearings over a pair of 24-in. reaction beams. Loading yokes composed of steel rods and built-up connecting beams were placed at the two load points, and loading applied by means of calibrated hydraulic jacks connected to a hand-operated oil pump and fitted with 12-in. Bourdon pressure gages. The loads were applied in increments of slightly less than 7,500 lb., and deflections were read after each load increment.

Effect of Reinforcement—A study of the data given in the table is indicative of the following facts:

(a) That for 1 per cent reinforcement, the average unit flexural stress does not materially differ from the compressive strength of the companion cylinders.

FIG. 2—LOADING SET-UP for tests included reaction beams, loading yokes and hydraulic jacks.



(*b*) That a marked increase in flexural stress is observable if the amount of reinforcement is increased to 2 per cent.

(*c*) That above 2 per cent there is a further increase in flexural strength but at a smaller ratio.

The variation between flexural resistance and standard cylinder strength is to be expected when it is remembered that the compressive strength of concrete cylinders is not fixed but arbitrary, being roughly an inverse function of the height ratio of the cylinder tested. The comparatively low flexural resistance of the beams with 1 per cent reinforcement is also to be expected in the light of the fact disclosed in the table that all beams with this percentage of reinforcement failed in tension.

Pure compression failures occurred only in the case of reinforcement percentages of 3.73 and above; however, the 2 per cent reinforcement produced a marked increase in flexural resistance because of its tendency to develop the concrete in compression.

It would appear from these tests that by increasing the amount of reinforcement from 1 to 2 per cent, an increase in flexural resistance of 34 per cent may be expected. If the amount of reinforcement is stepped up to 4 per cent, an increase of about 65 per cent results.

Effect of Web Reinforcement—Fig. 3 indicates a comparison of breaking strengths for the various beam types. The type 1 group, which contained no web reinforcement, did not appear to develop any increased flexural resistance as the percentage of reinforcement was increased, which obviously would be expected when it is observed from the table that failure in the case of this type of beam occurred by either direct or diagonal tension in every case.

The type 2 group, which contained bent-up bars, developed greater flexural resistance, although in this case also the failures were all in direct tension or diagonal tension.

Types 3 and 4 contained sufficient web reinforcement to develop failures in pure compression for the higher percentages of reinforcement.

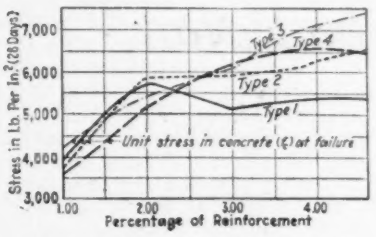


FIG. 3—EFFECT on flexural resistance of various types of web reinforcement.

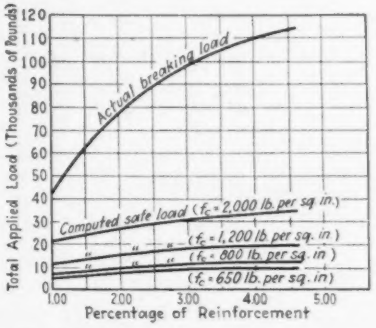


FIG. 4—COMPARISON of actual breaking loads with theoretical safe loads for varying unit stresses.

For the beams with 1 per cent reinforcement, the failure in each case was in pure tension, for which reason the manner of reinforcing the web appeared to have very little effect upon the ultimate strength. Where the amount of reinforcement was increased to 2 per cent, there did not appear to be enough tension steel to develop the efficacy of stirrup reinforcement, so that in this case also very little benefit was derived from a heavy reinforcement of the web.

Unit Stress Specifications—Fig. 4 is a comparison of actual breaking loads, with theoretical safe carrying capacities for assumed unit working-stress values of 650, 800, 1,200 and 2,000 lb. per sq. in. The rather extraordinary margin of safety would indicate that the lower values assumed for unit working stress are perhaps unduly conservative.

Conclusions

Based upon the foregoing discussion, the following tentative conclusions may be offered:

1. A great increase in load-carrying capacity for beams of this type may be obtained by increasing the reinforcement from 1 to 2 per cent, and a still greater increase, although at a less ratio, may be obtained by a further increase in reinforcement percentages.

2. This last fact may or may not be of importance to the designing engineer, depending upon conditions. In most cases economy will dictate the adoption of a low percentage of reinforcement with deeper sections and lower unit working stresses. However, where it becomes important to reduce dead loads or where clearances are so limited that shallow construction becomes particularly desirable, the results indicate possibilities in heavily reinforced slabs or shallow beams designed with a comparatively high unit working stress.

3. The necessity for adequate web reinforcement increases as the percentage of reinforcement is increased, in order that failure by diagonal tension shall not limit the ultimate capacity of the structural member.

4. The wide margin of safety disclosed by all of these tests points to the necessity for further study. Bearing in mind the fact that these beams were cast under field conditions, it would appear that a standard unit working stress in flexure of 1,000 to 1,200 lb. per sq.in. may safely be adopted for ordinary reinforced-concrete beam construction, and that in those cases where it becomes desirable to use heavily reinforced sections even higher working stresses may be safely employed.

The tests were conducted under the supervision of G. S. Paxson, assistant bridge engineer, and Jos. A. Weber, mechanical engineer for the Oregon state highway department.

ULTIMATE LOADS AND UNIT STRESSES OF BEAMS WITH VARYING AMOUNTS OF REINFORCEMENT

Beam Type	Data	$p = 0.0102$ 8—1″ ϕ = 2.4544 sq.in.	$p = 0.0200$ 8—1″ ϕ = 4.8104 sq in	$p = 0.0295$ 9—1″ ϕ = 7.0686 sq.in.	p 0.0373 9—1¼″ ϕ = 8.9460 sq.in.	$p = 0.0460$ 9—1¼″ ϕ = 11.0448 sq.in.
				$n = 8$		
		kd = 3.30 in. jd = 8.90 in	kd = 4.28 in. jd = 8.57 in.	kd = 4.90 in. jd = 8.37 in	kd = 5.30 in. jd = 8.23 in.	kd = 5.65 in. jd = 8.12 in.
1	Ultimate load, lb.	43,807	81,675	81,675	90,882	96,542
	Cyl. strength, lb./in.	4,270	4,250	4,532	3,665	3,870
	Unit stress conc.	3,920	5,720	5,116	5,338	5,382
	Unit stress steel	63,200	61,070	42,550	37,940	33,040
	Unit shear	111	208	213	240	257
				Diagonal Tension Failure (Steel)		
2	Ultimate load, lb.	40,837	83,160	94,446	103,600	117,315
	Cyl. strength, lb./in.	3,760	4,092	4,370	.....	4,337
	Unit stress conc.	3,670	5,820	5,894	6,067	6,515
	Unit stress steel	59,130	62,150	49,030	43,130	39,990
	Unit shear	104	211	244	272	311
3	Ultimate load, lb.	47,520	77,220	96,525	118,800	133,650
	Cyl. strength, lb./in.	4,052	4,240	3,707	4,342	3,570
	Unit stress conc.	4,240	5,420	6,021	6,938	7,406
	Unit stress steel	68,303	57,830	50,070	49,320	45,460
	Unit shear	120	197	249	310	352
			Tension Failure (Steel)		**Compression Failure (Concrete)**	
4	Ultimate load, lb.	39,650	72,765	98,110	111,375	115,830
	Cyl. strength, lb./in	3,870	3,965	4,087	3,785	4,087
	Unit stress conc.	3,580	5,120	6,123	6,518	6,439
	Unit stress steel	57,670	54,660	50,930	46,330	39,530
	Unit shear	101	186	253	291	307

Letters to the Editor

Bearing Capacity of Clay Soils

Sir—I have read with a great deal of interest your report of the discussion of soil mechanics (*ENR*, July 11, 1935, p. 45) which took place at the annual meeting of the Society for the Promotion of Engineering Education in Atlanta, Ga., June 24-25. I believe that your report brings out very well two somewhat conflicting conceptions of soil resistance that should be fully discussed. I do feel that the limited discussion that was possible at that time could not bring out clearly the essential differences of opinion, and I am taking this opportunity to clarify several points mentioned. For example, it is stated, "Professor Housel brought out that, in the methods used by these two speakers (Mr. Phillipe and Professor Casagrande) the possibility of exceeding the bearing capacity of the soil was never taken into account."

My position in this matter was rather that the consolidation tests on confined samples as presented by Professor Casagrande did not take into account the definite bearing capacity controlled by the ultimate resistance to lateral displacement. In Figs. 1 and 2 are shown the relations which, in my opinion, control the ultimate bearing capacity of a body of soil. Fig. 1 shows the stress reactions developed by the soil when subjected to direct load, the bearing capacity p being given by the linear equation shown.[1]

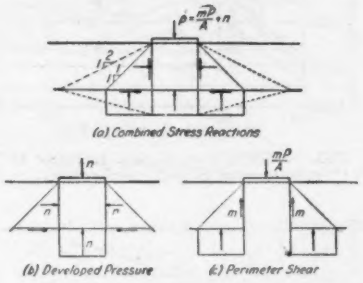


FIG. 1—STRESS REACTIONS developed by soil under direct load.

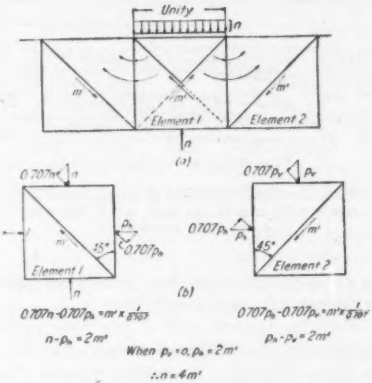


FIG. 2—RESISTANCE to lateral displacement as shown by the relation between shearing resistance and the ultimate value of the developed pressure.

When the boundary forces (perimeter shear) are properly evaluated, the resisting pressure developed by a body of material may be accurately determined and becomes the controlling factor in ability to support loads. As indicated in Fig. 1, the developed pressure n, while definitely related to volume change or consolidation, is dependent also on the confining influence of the surrounding body of soil. Thus the limiting value of the developed pressure or the bearing capacity limit is controlled by resistance to lateral displacement, which in turn depends upon the shearing resistance of the soil. If any loaded area stresses the body of soil in excess of its plastic limit, progressive settlement must follow due to lateral displacement. Very little, if any, of this progressive settlement can be charged to consolidation, and consequently a decreasing rate of settlement and eventual equilibrium appear to be a vain hope.

Fig. 2 shows the relation between shearing resistance and the ultimate value of the developed pressure, in which it appears that the critical value of the developed pressure is four times the ultimate shearing resistance or plastic limit of the soil. These theoretical relations have been verified by a correlation of load tests and independent tests of shearing resistance in a number of cases that were presented by the writer at the Atlanta meeting. The shear tests were made in the laboratory and by the penetration method, which has been described elsewhere.[2]

In connection with Mr. Phillipe's discussion, he made it clear that he considered the ultimate shearing resistance or plastic limit in his work and their designs were within this limit. There is no difference of opinion as far as the writer is concerned with this procedure. In many cases it may not be feasible to make direct load tests, and shear tests are adequate for determining ability to support direct load, providing the relation between shearing resistance and ultimate bearing value is known and recognized. It has been one of the primary objectives of investigations conducted by the writer to establish this relation and verify it by experimental proof.

Professor Casagrande replied, "emphasizing that in his own work the possibility of plastic flow or rupture in the soil is carefully investigated whenever it is required. He stated that in most problems connected with building construction the danger of failure of the soil is very remote, and that loading of the soil must often be limited to a very small fraction of its ultimate bearing capacity in order to keep the settlements small enough so they will not harm the building."

In the first place, Professor Casagrande's recognition of the possibility of plastic flow being carefully investigated is somewhat confusing in the light of previous conclusions he has presented. In an article in the *Journal* of the Boston Society of Civil Engineers, April, 1932, p. 207, he states, "A definite bearing value of clay does not exist. The engineer must learn that the kind of question he asks regarding the properties of clay underground should not be, 'What is the bearing capacity or the bearing value of this clay?' His question should be, 'How must I design my foundation so that the rate of settlement under the given building load will not exceed certain limits?'" On p. 191 of the reference cited Professor Casagrande presents, "Excavation—the only reliable method for reducing settlement."

In other words, the only means of limiting the settlement of a building is said to be flotation. There is no recognition at any point in this article of the importance of shearing resistance and plastic flow in the problem of bearing capacity, and these conclusions appear to represent the mature results of applying the consolidation theory to the design of actual substructures.

The second question to be considered in Professor Casagrande's discussion is whether or not the plastic limit of a soil is so great that it is of no practical importance. There is some confusion as to what is meant by rupture, which term is used more or less synonymously with plastic limit. Plastic materials are by definition materials which do not rupture but fail by progressive deformation or plastic flow. We would never expect them to fail suddenly when the ultimate shearing resistance is exceeded, unless the stress is very much in excess of the plastic limit. Such a rapid, sudden failure, similar to a landslide, probably would require a load out of the range of ordinary bearing pressures. In the writer's opinion, however, it is a serious error to ignore the entire range of plastic flow, which is much less in terms of load and which does in many instances produce progressive settlement that cannot be countenanced in the design of structures.

In the accompanying table are listed the results of fourteen series of bearing-capacity tests on various clay soils indicating the settlement and the developed pressure at the bearing-capacity limit or point of progressive settlement. For all materials tested, with two exceptions, the settlement at the ultimate bearing capacity was less than 1 in., and in all except three cases the safe bearing value selected for design was less than would have been used had the tests not been made. The behavior of

structures that have been built in accordance with these test results has shown satisfactory agreement with the anticipated settlement, and such results as are available have been presented elsewhere.[3]

Test Series	Settlement, In.	Developed Pressure, Lb./Sq.Ft.	Shape of Plates	
1	0.8	3,100	Round	1-9 sq.ft.
2	0.46	2,200	Square	Same location as 1
3	2.05	5,650	Round	
4	2.0	5,580	Square	Same location as 3
5	0.6	2,300	Round	
6	0.4	2,150	Square	Same location as 5
7	0.6–1.2	14,400	Round	
8	0.25	3,500	Round	
9	0.65	4,700	Round	
10	0.48	4,920	Round	
11	0.35	1,350	Round	
12	0.6	3,010	Round	
13	0.45	10,000	Round	
14	0.6	8,300	Round	

In the writer's opinion the specific evidence presented makes it perfectly clear that a definite bearing value identified as the bearing-capacity limit or plastic limit does exist and may be determined by load tests. Further, the evidence shows that this limit is in the range of ordinary bearing pressures and must be considered in the actual design of substructures.

Ann Arbor, Mich., July 17, 1935.

W. S. HOUSEL,
Assistant Professor of Civil Engineering, University of Michigan.

REFERENCES

[1]"Bearing Value of Clay is Determinable"—*ENR*, Feb. 23, 1933, p. 244.
[2]"A Penetration Method of Measuring Soil Resistance"—38th Annual Meeting Am. Soc. for Testing Materials, June 24-28, 1935.
[3]"A Practical Method for the Selection of Foundations"—Eng. Res. Bulletin 13, Univ. of Mich., October, 1929.

Computing the Spillway Flow at Boulder Dam

Sir—We wish to take the liberty of replying to the article entitled "Additional Data on Model Tests for Boulder Dam Spillways," by D. C. McConaughy, senior engineer, U. S. Bureau of Reclamation, Denver, Colo., which appeared in *Engineering News-Record* of April 4, 1935.

Mr. McConaughy appears to regard our article, "Analysis of Boulder Dam Spillways made by Swiss Laboratory," published in *Engineering News-Record* of October 25, 1934, as being a criticism of the design actually used. This was far from being our intention; we did not even possess the information necessary to enable us to make such a criticism.

The purpose of our article was rather to show, by giving a practical example, the accuracy of the formulas derived by our laboratory for computing the water surface in channels carrying varying quantities of water.[1] If we selected the Boulder Dam spillways to demonstrate this, it was because Boulder Dam is an undertaking of such extraordinarily large dimensions that merely for this reason it deserves to interest engineers all over the world. But there was certainly not the slightest intention on our part of submitting a counterproject.

Mr. McConaughy's article contains a few remarks with which we cannot agree and which we would therefore discuss here.

1. There is not the slightest doubt that the method of calculation of the Zürich Laboratory has given results more accurate than Mr. Hinds' method does. In order to make this clear, it is sufficient to regard the longitudinal profiles of the computed and measured water surfaces in Figs. 1A and 1B of Mr. McConaughy's article and in Fig. 2 of ours. The velocity of the water flowing in from the side has a component u parallel to the direction of the main flow, and if this component is neglected, as is done in Mr. Hinds' formula, systematic differences arise between the computed and measured water surfaces. The frictional losses are in fact not considered in Mr. Hinds' formula. In his article (*Trans.*, Am.Soc.C.E., Vol. 89, 1926) this is expressly pointed out. A factor h_f is introduced only to correct for friction in the numerical evaluations in Tables 2 and 3. But the method of computing this factor is not shown. Besides that, the correcting factor is improperly introduced in the tables; h_f should not be added to the difference in the water surfaces but to the difference in the heights of the energy lines.

2. The design actually used provides a cross-weir or step across the channel at the lower end, which, according to what Mr. McConaughy states, is intended to steady the flow before the water enters the shaft. We have no objection to make to this, provided that the extra expense of the greater excavation work thereby rendered necessary is considered justifiable. But on the other hand we doubt if it will have any considerable influence on the quantity of air carried over in the water, since this is a phenomenon dependent on other factors, as we point out more in detail below.

3. The shape and height of the gates has not been studied by us. The problem that concerned us was simply, with a given height of crest at El. 1205.4 (with the gates completely open) and a prescribed maximum water level at El. 1232, to let off the prescribed quantity of water.

From Fig. 1C of Mr. McConaughy's article it appears that the discharge from the two upper gates in the final design is to a great extent partly submerged. We therefore doubt if the quantity of water in the tests described by the author is the same quantity as that on which our tests were based.

4. Concerning the shape of the piers, our remarks referred to the faces of the piers directed downstream, since the faces directed upstream are in any case given a favorable rounded shape. At the downstream faces of the piers the discharge is torrential, so that the shape of the piers at that spot is of no consequence.

5. Although the outlet of the tunnel opens in free air, in our tests it was found that the flat part of the tunnel comes under pressure. The axis of the outlet was then taken as being at El. 665. A hydraulic calculation that takes no consideration of the air included in the water certainly shows that the flat part of the tunnel should not, under this supposition, come under pressure. But then the water would attain speeds of over 160 ft. per second. Now, experience with very steep flow-off channels shows that very much air is then included in the water. The consequence of this is that the upper surface of this mixture of water and air is considerably raised and that the velocity of flow is reduced. Besides that, this phenomenon is independent of the more or less great turbulence of the water before entering the shaft leading to the tunnel, and depends on the velocity of the water. When a steeply inclined shaft leads to the tunnel and is initially not quite full, a very strong current of air will be caused, the water will absorb air and occupy in the tunnel a greater volume than given by a computation that neglects the air; the tunnel will therefore soon work under pressure.

Zürich, Switzerland, July 24, 1935.

E. MEYER-PETER
HENRI FAVRE
Hydraulic Laboratory, Ecole Polytechnique Fédérale.

[Mr. D. C. McConaughy, to whom the above letter was submitted, replies as follows:]

Sir—I have read with much interest the reply of Drs. E. Meyer-Peter and Henri Favre to my discussion of their original article in *Engineering News-Record* of April 4, 1935.

Most of the points raised concern matters of fact, and I believe that enough data have been published to enable an interested reader to form his own conclusions. Beyond assuring the authors that there was no intention of decrying their formula, which represents an advance, the writer has no comment to make.

D. C. MCCONAUGHY.

San Marco's Activated-Sludge Plant

Sir—Professor Steel's interesting article entitled "Pioneer Activated-Sludge Plant Still in Service" (*ENR*, Aug. 15, 1935, p. 222) leaves indefinite the date that the plant was put into operation. This was given as "about Sept. 1, 1916," in an article by Henry E. Elrod in *Engineering News*, Feb. 8, 1917, p. 249. Disposal on land of the excess sludge from this plant pioneer was described by E. G. Eggert and Chester Cohn in *Engineering News-Record*, May 23, 1929, p. 830.

Upper Montclair, N. J., Aug. 22, 1935.

M. N. BAKER.

[1]H. Favre: "Contribution à l'Étude des Courants Liquides," published by Rascher & Cie., Zürich, 1933. (Formula (1) of our article of Oct. 25, 1934, contains a printer's error, the minus sign having been omitted before Δ Z).

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September 19, 1935

Road and Crossing Work Speeded

THE HIGHWAY and grade-crossing elimination program received substantial aid through last week's developments. The $1,400-per-man-year limitation was entirely removed from grade-crossing work, and the highway regulations were modified to permit each state to set up such part of its total program as it is able to finance without waiting until it can finance its entire program under the $1,400 limitation. Elimination of the labor restriction from grade-crossing work has been urgently needed, for the $1,400 limit has effectively held up a large amount of useful work and has forced consideration of makeshift plans in which the ratio of labor to material costs could be greatly increased. That a modification of the highway regulations also was much needed is apparent when it is realized that only two states have had their allotments approved. Heretofore, each state had to adjust its entire federal-aid highway program so as to make the average come within the $1,400 limit. Now by putting up enough of its own funds to bring the labor costs chargeable to federal appropriations down to the prescribed limit, a state can undertake any part of its program. Thus high-cost work near centers of unemployment can be put under way quickly without waiting development of the whole program. The change should result in the release of a large amount of highway work.

The Result of Dilly-Dallying

THE ICKES-HOPKINS CONTROVERSY over the type of projects to be favored in alloting the remaining funds under the Works Relief Act was settled last week in Mr. Hopkins' favor. Only such PWA projects will be financed as can be put under contract by Dec. 15, can be completed within a year and will not cost over $850 per man-year of employment. This is indeed a disheartening decision for the hundreds of city officials as well as engineers in private practice who have worked so hard to get together the mass of data required to put PWA projects into shape for approval. And no amount of fine words now as to the urgency of getting men off the relief rolls and the inherent slowness of large public-works projects in creating employment will conceal the fact that the present impasse is due to the dilly-dallying of the Works Progress Administration. The 1935 Relief Act was signed on April 5, a month was wasted in setting up the complicated system of approvals that has done so much to check rather than speed progress, rules for filing applications for funds were not announced until mid-May, the now-famous $1,100-per-man-year limit was not announced until mid-June, and the first allotments were not approved until the end of June. That record has continued through the summer. PWA projects were among the principal sufferers. Many believe that they were deliberately delayed, to create a situation in which they would have to be scuttled. Had they been acted upon promptly, there would now be no question of getting a vast number of them under contract by the middle of December.

From PWA to WPA

ATTEMPTING TO VIEW the present works-relief situation constructively, this should be said: We at least have something definite to act on in the knowledge that expenditures hereafter authorized are to be concentrated on small WPA projects. Such being the case, every effort should be directed toward making those projects productive of the greatest possible amount of permanent public benefit. Much work that could have been handled as PWA projects—and should have been so handled—can be made over into WPA projects, as has been done by some cities that were unable to finance PWA projects. Paving, sewer and water-pipe laying, for example, can be split up into short sections, to keep the cost of each project within the maximum limit set for such work. This calls for a quick review of the useful work that is available.

A Lesson in Coordinated Effort

THREE MONTHS AGO the Supreme Court, by its decision in the Schechter case, wiped out the code section of NIRA and its industry codes. In the ensuing readjustment of business planning the original purpose of industrial self-government which the codes were intended to make possible was speedily forgotten, and little or no thought was given to what was learned in code administration. As future progress is certain to be guided largely by code experience, a record of successes and difficulties encountered under the construction code is fundamentally important to all elements of engineering and construction. James W. Follin, who for two years was intimately associated with the planning and administration of the construction code, has prepared that record, the first half of which appears elsewhere in this issue. It shows that despite the reverses, nullifications and difficulties, the code experience served to emphasize the advantages of a coordinated industry.

Investment and a Corollary

WHAT IS THE TRUTH about the oft-repeated charge that too much money was saved and put into productive plant during the busy years of the twenties? A good many people have come to believe, by hearing the charge reiterated, that in those days not enough of our total production was turned into consumptive channels by way of worker income, and that too much flowed into fixed investment. Illuminating data bearing on this charge were recently brought together by Virgil Jordan, well-known statistical researcher and president of the National Industrial Conference Board. They show that the collapse of 1929 and the ensuing depression cannot be so explained. Dr. Jordan finds that the percentage of our income going to investment in new productive plant was measurably constant during more than twenty years, and if anything it was smaller in the later years. Valuable as this finding is in itself, it has an even more valuable corollary—namely, that the normal rate of construction investment during the twenties, some eight billions per year, was in no sense too high but barely succeeds in keeping pace with the natural growth of requirements.

Rail-Fissure Research

FOR MORE than a quarter of a century railroad maintenance engineers have been baffled by an ever-increasing number of rail failures called by the section hands who report them "clean breaks," because to the untrained eye the fracture was a fresh break as compared with the rusted fracture resulting from a piped rail. Occasionally, however, more careful observers noted a round and slightly rusted area in the head of some of these fractured rails, the rusted area extending to an edge of the rail, and then still more careful observers noted that freshly broken rails revealed similar rounded areas that had a peculiar brightly polished surface. In 1911 the late James E. Howard identified these failures as a distinct class, quite different from the clean breaks that occasionally occur in rails, especially during periods of low temperature, due to a blow from a flat wheel or other external cause. The cause here appeared to be internal, the progressive development of what is now known as an "internal fissure."

WHAT caused fissures and how or why they developed remained a mystery. Once recognized, they were found in rails of length of service varying from a few weeks to many years. They were found in certain heats and in the rollings of one rail mill during one year more than during another. Evidence accumulated by the railroad companies and the American Railway Engineering Association pointed to more frequent occurrence in densely traveled track, thus indicating that development of the fissures to the point of causing failure was a function of traffic density or of wheel loads. Mr. Howard early developed the theory that the fissures were due to the cold-rolling of the rails in service, but there was sufficient contradictory evidence to make general acceptance impossible. What was clear was that the fissures grew in area progressively until failure occurred or the rail was removed from service for other cause. This resulted in a long controversy between rail manufacturers on the one hand and railroad men on the other, the manufacturers maintaining that the fissures were due to high bending stresses in service and the railroad men maintaining that high track stresses had little to do with the origin of fissures. Meanwhile the joint committee on stresses in railroad track, under the leadership of Arthur N. Talbot, was producing new knowledge on the magnitude to which track stresses may develop.

A correlated study, sponsored by the A.R.E.A., but one having no bearing on the determination of the cause of transverse fissures, led to the design of the Sperry cars for locating transverse fissures in the track. Detection of fissures by these cars served to minimize the risk and yet to amplify the evidence of the seriousness of the problem facing the railroad men.

FOUR YEARS AGO railroads and rail manufacturers undertook an exhaustive study to determine the cause of transverse fissures. Now the first information as to what has been accomplished in that time has been made public in a bulletin of the A.R.E.A., summarized in this issue by H. F. Moore, who was in charge of the test parties. The studies have been highly productive in bringing out information on the cause of the formation of fissures, but some questions remain to be answered.

In the first place the studies fully confirm the earlier conclusions that the *development* of fissures is the result of repeated stress. They showed that in the laboratory transverse fissures can be developed under a rolling load in rails known to have "shatter-cracks," a microscopic defect occasionally found in steel rails, and that they could not be developed in rails free from such cracks. However, fissures are not developed in *all* rails that have shatter cracks in them. The limiting load, which determines whether a rail will fissure before it wears out, depends not only on traffic conditions but also on the direction of any shatter-crack and its distance below the tread of the rail.

THE TESTS showed a close relation between intensity of internal stresses directly under the wheels and the development of fissures; also that these stresses are practically independent of the size of rail. This again confirmed service observations. Fissures have developed alike in large or small rails. Of special significance was the finding of evidence of a reduction in the fatigue limit of about 50 per cent in cracked steel as compared with uncracked steel. Field observations showed that frequently track or rolling-stock conditions resulted in stresses higher than those required to start the development of fissures in rails having shatter-cracks.

Turning to manufacturing processes to see what could be done to reduce or eliminate shatter-cracks, the studies indicate that a very definite reduction in such defects can be obtained by thermal treatment, using commercially practical processes. Entire elimination of the cracks has not been obtained with all of the processes tested. On the other hand, in the laboratory tests it was not found possible to develop fissures in thermally treated rails by the rolling-load test.

Thus it is apparent that the studies already have gone a long way toward the elimination of transverse fissures by showing, first, that they develop from shatter-cracks in rails and that these cracks can be reduced in number, if not entirely eliminated, by thermal treatment of rails as they come from the rolling mill; and, second, that the development of cracks in service can be reduced by improved rolling stock and better track. These results can be made immediately productive of greater safety and less loss of service life in rails. Continuation of the studies, especially those of manufacturing processes, can be expected to lead ultimately to the elimination of transverse fissures as a major cause of rail failures.

FROM another angle this research is significant; it is the first penetrating approach to a problem of twenty years' or more standing. The productive results obtained with so little lost motion can be laid to two things; first, to the investigative acumen of those engaged to direct it; and second, to the revelations of an earlier and equally penetrating study directed toward a related problem, that of the track stress committee. The latter pointed an accusing finger at locomotive unbalance and high wheel loads as prime sources of track troubles. Working from the evidence of high stress in the rail recorded by that committee, and benefiting by its experience in developing devices for measuring strains, those in charge of the present study moved rapidly toward fuller revelations of the fatigue conditions set up in the head of rails in service. The present study strongly reinforces Professor Talbot's statements concerning the importance of track maintenance, strongly emphasizes the importance of keeping rolling stock (especially car wheels) in first-class condition, and raises a serious question as to the safety of any further increase in wheel loads, either locomotive or car-wheel loads.

There could be no more impressive demonstration of the value of properly directed research.

CURRENT NEWS

Bids Asked on First Section Of Sixth Avenue Subway

Bids for the construction of the first section of New York's Sixth Avenue subway from 40th to 47th Streets will be opened on Oct. 4 by the Board of Transportation. The contract provides that the work shall be done during such hours of the day as will cause a minimum of inconvenience to business and traffic along the avenue. Sixth Avenue trolley service will be discontinued entirely in the construction areas, and during the day the open-cut sections are to be decked over, but after midnight the avenue may be completely closed to traffic. Columns of the Sixth Avenue elevated railroad are to be supported without interruption to that service. The completed subway will run from the Queens line at 53d Street to 4th Street where it will connect with the 8th Ave. subway.

Much of the work will be in rock areas, eliminating the necessity of disturbing the surface. The line will be four tracks wide, except at the 53rd Street connection, where the tracks will be on a graduated level.

The roadbed will be as deep as 60 ft. at some points. Due to the water tunnel 220 ft. below the surface of Sixth Avenue, the rock blasting for the new subway will be confined to light charges of dynamite.

Construction of the subway is being financed by the city.

All Engineers in Wisconsin Now Must Register

By an amendment to the Wisconsin registration act approved on Sept. 5, architects and all branches of engineering are included under the act. The fee is reduced from $15 to $10. The law as amended does not require that professional engineers be classified although that was proposed, the sentiment of the existing Board of Examiners being against classification. This board of nine members includes the state architect, the state engineer, the dean of the college of engineering of the University of Wisconsin, three architects and three professional engineers. The present personnel will continue to function, holding office until their terms expire under the old act. Appointments are made by the Industrial Commission upon recommendations from the architectural and engineering societies of the state.

Professional engineering defined

The practice of professional engineering is defined in the act as: "Any professional service, requiring the application of engineering principles and data, wherein the public welfare or the safeguarding of life, health or property is concerned and involved, such as consultation, investigation, evaluation, planning, design, or responsible supervision of construction, alteration, or operation, in connection with any public or private utilities, structures, bridges, industrial plants, buildings, machines, equipment, processes, works and the structural members of other than industrial buildings."

No Funds Provided For Carrying Out Parker Dam Work

THE FACT that no funds have been allotted for continuing the work at Parker Dam was recently pointed out to the Metropolitan Water District of Southern California by Elwood Mead, Commissioner of Reclamation, while in Los Angeles. It was expected that the work would be carried out with funds from the $100,000,000 allocated to the Bureau of Reclamation in July. The allocation funds for reclamation projects is listed in *EN-R*, July 25, 1935, p. 133. No explanation as to why funds for Parker Dam were not set aside has been made.

If the water district is unsuccessful in getting Secretary of the Interior Ickes to reallocate the funds, it will have to finance the project by the sale of bonds to private investors or as collateral for an RFC loan.

Up until the time work was stopped, on April 29, 1935, by a U. S. Supreme Court decision that the work was unauthorized, construction was being carried out under a PWA loan and grant of $2,000,000. The Rivers and Harbors Bill recently signed by President Roosevelt carried specific authorization for the project, and work was ordered resumed on Aug. 31. Plans for the development of the Gila Valley reclamation project call for the use of power developed at Parker Dam.

Contract awarded for Cajalco Dam work

The Griffith Company, Los Angeles, Calif., has been awarded the contract by the Metropolitan Water District of Southern California for the construction of Cajalco Dam, a unit of the Colorado River aqueduct storage and distribution system. The company's bid was $4,646,856. Cajalco Dam, to be built ten miles north of Riverside, Calif., will be approximately one-half mile long and 185 feet high and will store 100,000 acre-feet of water. The lake to be formed by the dam will be the main reservoir and western terminus of the aqueduct system.

District bonds to be sold

The Metropolitan Water District of Southern California will receive bids on Sept. 20 for $12,096,000 Colorado River waterworks bonds. The bonds are to be dated Oct. 1 and will mature $336,000 annually each Oct. 1 from 1950 to 1985, inclusive. Bidders are asked to name coupon rate on the bonds not to exceed 5 per cent. To date the district has $55,000,000 of the bonds outstanding, all but $1,500,000 of which have been purchased by the RFC at 5 per cent. The $1,500,000 went to the PWA at a 4 per cent rate. The district is authorized to issue $220,000,000 of the bonds for the construction of the project. These bonds are to be used for work other than the completion of Parker Dam. No plans for financing the completion of the work at Parker Dam have been developed as yet.

Chicago Gets WPA Funds For Street Work

To repair, resurface and regrade more than 1,000 mi. of streets in Chicago, WPA has allotted and controller general J. R. McCarl has released $32,543,000. Work started Sept. 16 with a transfer of 900 men from relief rolls to WPA rolls. Ultimately 38,000 men will be employed on the projects.

The city's share in the cost of this project is $5,000,000 of which $1,500,000 is a credit for the rental of the three municipal asphalt plants, truck rollers and other equipment as well as $1,000,000 credit for brick and $300,000 for stone obtained from the Sag Channel spoil banks. As a further contribution the street department will furnish 2,100 employees at prevailing wages. Some 1,800 city employees in the street, water pipe extension and sewer departments were laid off Sept. 12 and are to be reemployed on WPA projects as supervisors, foremen and sub-foremen.

Costs per man-year were estimated at $1,020. The average per square-yard costs range from $2.02 to $2.61.

Engineering survey parties are first to ascertain and find the exact location of every different kind and yardage of pavement to be rehabilitated. Draftsmen will prepare zone maps indicating just what is to be done at each location. This work is divided into three divisions of 30, 30 and 70 zones each. Later new ward maps are to be made showing the rehabilitated surfaces, underground utilities, curbs and gutters.

The total yardage to be improved equals 1,118 miles of 30-ft. street. Total wages are estimated at $24,600,000 and materials at $11,400,000, made up principally of the following items: 450,000 tons of asphalt top at $6 per ton, $2,700,000; 750,000 tons binder at $4.50 per ton, $3,375,000; 578,000 brick, $1,000,000; 1,139,000 cu. yd. crushed stone, $578,000; 258,000 cu. yd. of sand, $323,000; 962,500 bbl. of cement, $2,311,000; miscellaneous, $1,140,000. Material costs it is stated have been substantially reduced below what was thought possible by employing relief labor in the fabrication of such material as asphalt.

Santa Ana, Calif., Will Vote on Flood Control Bonds

The approval of bonds amounting to $6,620,000 will be sought by an election at Santa Ana, Orange County, Calif., on Oct. 4. The bonds if approved, will be used to aid in financing a flood control project already approved by the War Department, and for which an allotment of $6,374,000 of federal funds will be made.

Included in the flood control work will be Prado Dam, $7,215,397; Brea Dam and channel, $697,943; San Juan Dam, $1,077,980; Carbon Canyon Dam and conduit, $623,282; Fullerton Dam and diversion channel, $149,737; Trabuco Dam, $617,500; Aliso Dam, $110,850; Santiago Dam and levees, $498,560; Santa Ana River channel, $375,000; Brea pipeline, $86,108.

Reclamation Bureau Gets Allotment For Central Valley Work

AN ALLOTMENT of $20,000,000 toward the start of work on the $160,000,000 Central Valley irrigation and power project in California has been approved by the Works Progress Administration. The allotment is to be made to the U. S. Bureau of Reclamation. Walker R. Young, the bureau's engineer in charge of Boulder Dam construction is to be in charge of the Central Valley project for the Bureau of Reclamation. An outline of the Central Valley project was given in *ENR*, May 18, 1934, p. 619. No power plant is contemplated in the work now to be started. The proposed work includes a large amount of construction work such as:

1. Control of the San Joaquin River through construction of a dam 252 ft. high at Friant, to store 400,000 acre-ft. gross or 270,000 net effective, and to have a discharge capacity of 15,000 sec.-ft. plus 92,000 sec.-ft. spillway capacity.

2. Control of the Sacramento River through construction of a dam for which a site near Kennett has been suggested by the state.

3. Construction of canals to provide water for six counties in the southern end of San Joaquin valley where present supplies are being exhausted.

4. Construction of a conduit to provide fresh water from the Sacramento River to the industrial cities on the Contra Costa County shore of Carquinez Straits.

5. Construction of a cross-channel diversion by which Sacramento River water will be diverted to the San Joaquin channel where it will be pumped upstream through a series of shallow locks for use in the lower end of the San Joaquin valley.

Major problems to be solved by construction of the project are those growing out of the falling of the water table in the San Joaquin valley, where some 40,000 acres of highly developed farm and orchard lands have been abandoned, those growing out of infiltration of salt water from San Francisco bay into the delta region during stages of low water in the Sacramento River, and the flood and navigation problems arising from an unregulated river of erratic flow.

Concurrent development of both the rivers of the great inland valley of California alone will provide solutions for all these problems. The San Joaquin, draining the southern reaches of the Sierra Nevada, carries insufficient water for all the needs along its course. The Sacramento River, draining the northern half of the great mountain chain, has an overabundance of water but like all rivers in arid and semi-arid territory it is subjected to tremendous fluctuations.

To use waters from the San Joaquin on the drying lands of the southern counties, would deprive other agricultural areas farther down the stream of their supply, unless, as is proposed, waters of the Sacramento are diverted for their use.

To divert waters from the Sacramento without regulating its flow would aggravate the saline encroachment which has caused major damage in the delta and to the industrial cities of the straits area.

But by storing the spring floods of the Sacramento in a large reservoir, a steady head of water sufficient to force the salt water back into the bay, provide surplus for diversion into the San Joaquin valley, protect against flood, improve navigation of Chico Landing and generate a large amount of firm power can be provided.

The Bureau of Reclamation expects to begin work on all the major engineering features of the project with the allotment made by the WPA. In six months, the bureau expects to provide steady employment on the project for 9,000 men.

PROPOSED WORKS IN THE GREAT CENTRAL VALLEY PLAN

Item	Height of Dam (Ft.)	Capacity	Installed Capacity of Power Plant	Estimated Capital Cost
Kennett reservoir	420	2,940,000 ac.ft.	325,000 kva.	$84,000,000
Sacramento-San Joaquin delta cross channel	...	10,000 sec.ft.	.....	4,000,000
Contra Costa County Conduit	...	120 sec.ft.	.....	2,500,000
San Joaquin River pumping system	...	3,000 sec.ft.	.....	15,000,000
Friant reservoir	252	400,000 ac.ft.	30,000 kva.	15,500,000
Madera canal	...	1,500 sec.ft.	.....	2,500,000
San Joaquin River-Kern County canal	...	3,000 sec.ft.	.....	27,400,000
Water rights and general expense	...	.....	.....	8,000,000
Total				$158,900,000

THE CENTRAL VALLEY WATER PROJECT—Originally planned as a project to be undertaken by the State of California, will now be carried forward as a U. S. Bureau of Reclamation project to provide water supplies for domestic, municipal, irrigation and industrial uses, benefit navigation, furnish flood and salinity protection and develop hydro-electric energy.

Rio Grande Flood Control Given Approval

Recent approval by President Roosevelt of the House bill which provides for control of the flood waters of the Rio Grande for 100 mi. along its lower course makes possible a request upon the National Emergency Council for a $4,743,000 allotment to complete the lower Rio Grande Valley flood control project, upon which $2,000,000 already has been spent. The government also would assume maintenance of the control system and the protection of 100,000 acres of farm land. This maintenance, now borne by Hidalgo and Cameron counties, costs about $100,000 annually.

The bill authorizes the State Department to negotiate a treaty with Mexico for flood control work, division of Rio Grande waters, channel rectification, more nearly correct establishment of the international boundary and other items. The bill's provision for equitable division of the Rio Grande waters applies only to the portion of the river below Fort Quitman, a previous treaty having divided the waters above that point.

The report of government engineers also recommends, in connection with the flood control project, the construction of two dams on the Rio Grande. One of these would be located a short distance above Roma, Tex., at a cost of about $6,000,000. The other would be located near Laredo.

WPA Gets Most Of Remaining Funds To Carry Relief Work During Winter

(Washington Correspondence)

BALANCING time, men and money is a simple problem that was quickly solved by President Roosevelt. In order to provide employment for 3,500,000 men from relief rolls, President Roosevelt held that from December through March WPA will have to carry the bulk of the load. To the Hopkins organization, therefore, went most of the $1,250,000,000 which remained for allotment this week from last April's appropriation of $4,880,000.000, after pegging $85,000,000 more for direct relief (total now $800,000,000) and $75,000,000 for CCC (total, $597,000,000).

Countrywide protest against the slashing of present public works program may impel Congress to give the President specific instructions for the spending of further appropriations for relief work. The money allotted to PWA projects out of the last appropriation is less than half the $900,000,000 earmarked by the law but Ickes covers his discomfiture by explaining that this was only suggestive.

The Sept. 17 deadline for completing allotment of the entire $4,880,000,000 was ignored. With limits already set by the President on PWA, this becomes a perfunctory job which he may dispose of before returning to Washington on Sept. 24. In any event, the Advisory Committee on Allotments will not meet until after his return. The Dec. 15 deadline for getting all work under contract does not apply to PWA projects previously approved, according to Ickes.

Total of PWA projects high

Although PWA Administrator Ickes trailed in the Hyde Park match with Harry Hopkins, PWA's program will run over $835,000,000 in cost. Grants for PWA projects approved to Sept. 16 total $126,738,000. Approximately $200,000,000 will be added for PWA projects by cutting its housing program from $240,000,000 to $100,000,000 and pinching Tugwell's rural resettlement and Morris Cooke's rural electrification program. The President regards rural electrification as a simple business proposition which RFC should be able to finance. Upwards of $50,000,000 more will be found for new PWA work by recapturing allotments made for previous PWA projects which sponsors have failed to push for various reasons. All of this money, totaling $376,000,000, will be distributed as grants amounting to 45 per cent of the cost of the projects approved. Sponsors must add 55 per cent. This doesn't run so far short of PWA's original program of nonfederal public works which, after allowing for recision of projects which have been delayed in execution, reached a total of approximately $1,000,000,000, of which the government put up $670,000,000 in grants and loans.

In shattering Ickes' dream of a $2,000,000,000 public works program calling for $900,000,000 in grants for relief works funds, the President saved the PWA administrator's dignity by taking Hopkins' veto power over PWA projects into his own hands. Ickes submitted a list of projects this week calling for grants totaling $200,000,000. This was the cream of applications totaling $2,500,000,000 for grants or loans or both which PWA's drag net had brought in in the hope that the President would shear away from WPA's program of made work and boondoggling.

In accordance with the President's instructions, the man-year cost of these new PWA projects will not exceed $850, which brings them down practically to the WPA level. They are likewise such as can be put under contract by Dec. 15 and completed within one year.

The President is counting on PWA work to get into full swing by the time WPA's program tapers off next July but PWA obviously can't assume the entire burden. Another works program is already in sight to employ labor that private industry fails to absorb. The President is noncommittal on this, save for his significant remark at Hyde Park that such widespread improvements have been made in industrial technique that 1929 volume of production probably would not now require more than 80 per cent of the labor employed in that year. As a step which the administration is forced to take as an alternative to resuming direct relief, the next works program will be cast in the same mold as CWA and WPA.

The administration is committed to an impressive list of huge flood control, power, water supply and navigation projects that will require several hundred million dollars to carry to completion. The President regards $500,000,000 a year as desirable to carry on a normal program of public works but he is still confronted by an emergency that must be reckoned with in the budget now in the making for the fiscal year commencing July 1, 1936.

Regardless of whether another relief works program follows the present one there will be plenty of unfinished business in public construction after next June. Army Engineers estimate additional appropriations needed to complete works already started at over $400,000,000 as compared with normal annual expenditure for both construction and maintenance of $75,000,000. Fifteen unfinished reclamation projects need additional appropriations of $250,000,000.

The Tennessee Valley program is unsettled, so no definite requirement can be determined, but more money is necessary to finish Pickwick Landing Dam as well as the proposed storage dam on the Hiwassee River and four others under study.

Bids on Disposal Plant Asked By New Jersey Municipalities

Bids for the construction of a sludge disposal plant to serve eleven New Jersey communities in Essex and Union Counties will be received on Oct. 3, at the City Hall, East Orange.

The work on which the bids will be taken completes the sewage disposal system designed for the communities of East Orange, Hillside, Irvington, Maplewood, Milburn, Newark, Roselle Park, South Orange, Summit, Union and West Orange. Alexander Potter, consulting engineer, New York City, designed the sewage disposal system.

N. Y. Contractor Loses Suit Against State for Extra Costs

The New York State Court of Appeals has dismissed an $8,000 suit brought against the state of New York by the Feeney & Sheehan Building Co., Inc., who built the pedestrian tunnel connecting the basement of the State Capitol with the State Office Building. The contractors alleged breach of contract on the part of the state, contending that the discovery of quicksand beneath the new structure, the existence of a large concrete mat beneath the ground of the western approach to the Capitol and other unexpected developments necessitated changes in specifications during construction and caused increased costs.

The court admitted the contentions but held that the president of the contracting concern had previous knowledge of quicksand on Capitol Hill and that the presence of it could easily have been discovered upon an inspection. His previous knowledge of general conditions in and about the Capitol should have given him advance knowledge of the concrete mat, the decision declared. Quicksand was encountered in practically the entire length of the tunnel, according to the suit. The concrete floor of the tunnel disintegrated in certain sections several times during progress of the work, necessitating patching and repairing. The failure of the contractor to keep the excavation free from water during the laying of the sub-base caused this condition, according to the court's decision.

F. P. Gutelius, Former Railroad Engineer, Dies

F. P. Gutelius, resident vice president of the Delaware & Hudson Co. at Montreal, died at North Bay, Ont., on Sept. 12, following a long illness. Mr. Gutelius made his home in North Bay when general superintendent of the Lake Superior Division of the Canadian Pacific Ry.

Born in Mifflensburg, Pa., in 1864, and graduated from Lafayette College as a civil engineer in 1887, Mr. Gutelius began his railway career with the Pennsylvania in 1888. In 1894 he went into mining work in the west during which time he built the Trail Creek Tramway. Later he became chief operating officer of the Columbia & Western Ry., which he extended and on which he remained as superintendent when it was sold to the Canadian Pacific in 1898.

With the Canadian Pacific Mr. Gutelius rose rapidly through the engineering department to the position of assistant chief engineer, which position he vacated in 1908 to become general superintendent of the Lake Superior Division when extensive reconstruction of that division was undertaken. Later he occupied the same position at Montreal.

In 1912 Mr. Gutelius was appointed a member of the Canadian Government's commission to investigate the construction of the National Transcontinental Railway. He served with the commission until his appointment to be general manager of the Canadian Government Rys., now part of the Canadian National Rys. In 1917 he was appointed resident vice president of the Delaware & Hudson R.R. at Montreal and during the war period acted as federal manager of that railroad.

SOCIETY CALENDAR

AMERICAN WELDING SOCIETY, fall meeting, Chicago, Ill., September 30 to October 4.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, Milwaukee, Wis., October 7-10.

NATIONAL SAFETY COUNCIL, annual congress, Louisville, Ky., October 14-18.

PUBLIC WORKS ENGINEERING CONFERENCE, joint meeting of the American Society of Municipal Engineers and International Association of Public Works Officials, Cincinnati, Ohio, October 14-16.

INSTITUTE OF TRAFFIC ENGINEERS, sixth annual meeting at Louisville, Ky., October 15-16.

AMERICAN SOCIETY OF CIVIL ENGINEERS, fall meeting, Birmingham, Ala., October 16-18.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, White Sulphur Springs, W. Va., October 16-18.

CANADIAN INSTITUTE ON SEWAGE AND SANITATION, annual convention, Toronto, Ont., October 17-18.

NATIONAL SOCIETY OF PROFESSIONAL ENGINEERS, first annual convention, Columbus, Ohio, October 23-25.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual meeting, Columbus, Ohio, October 23-25.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual convention, Miami, Fla., December 9-12.

NEW ENGLAND AND NEW YORK STATE SEWAGE WORKS ASSOCIATIONS, joint meeting at Schenectady, N. Y., October 4-5.

NORTH DAKOTA WATER AND SEWAGE WORKS CONFERENCE will be held September 30-October 1 at Grand Forks. An operators' water and sewage school is scheduled for the second day. M. D. Hollis, Bismarck, is secretary.

WISCONSIN SECTION AMERICAN WATERWORKS ASSOCIATION, annual meeting, Sheboygan, Wis., November 4-6.

EXAMINATIONS for registration of structural engineers and land surveyors will be held in the near future as follows:

CALIFORNIA—By California Board of Registration for Civil Engineers at State Bldg., Los Angeles, and at State Office Bldg., Sacramento, October 24-25.

Denver Votes Approval of Water Project Bonds

Denver voters, on Sept. 10, voted approval of two bond issues to be used in carrying out water supply projects. One issue is for refunding $2,700,000 of 4 per cent bonds at 3¼ per cent. The other issue is to provide $2,700,000 for financing the Fraser River extension of the city's water supply. This amount will be supplemented by a PWA grant of $2,137,725.

The project is to divert water from the western slope of the Continental Divide through the pioneer bore of the Moffat Tunnel, and by a system of canals and conduits, to the Fraser River and thus into Denver. A filter plant and reservoir are to be included in the project.

Utah Law Requires Licensing Of All Water Wells

A new law recently enacted in Utah makes it mandatory that all claims for underground water rights be filed with the state engineer. Offices for filing the claims have already been opened as the law goes into effect on the first of the year. Anyone heretofore using underground water must make his claim to the state. A filing fee of $2.50 is made for each well. Well drillers are required to make a report within 30 days after completion of a new well, telling the location and flow.

MID-WEST'S SUSPENSION BRIDGE APPROACHES COMPLETION

Current suspension bridge operations in the United States have a wide geographical distribution. The East has its Triborough Bridge at New York, the West its Bay Bridge and Golden Gate Bridge, and the Middle West its Davenport Bridge over the Mississippi River. The Mid-West's bridge will be completed first, as this air photo indicates. Present plans are to open it to traffic late next month.

Connecting Bittendorf, Iowa (near Davenport), with Moline, Ill., the new bridge is being built as a public toll structure by the Davenport Bridge Commission with $1,750,000 by PWA funds. In the view, the narrow well-defined channel near the center is the old Rock Island barge canal which has not been used since the Rock Island dam two miles downstream was completed in 1933. On the nearest anchor pier of the suspension bridge may be seen the widened roadway where toll houses and administrative quarters will be located. The main suspension span is 740 ft., the side spans each 370 ft., and the over all length of the bridge is 5,552 ft. Sixty feet of clearance above high water is available.

The task of placing the concrete for the 24-ft. roadway and the 4-ft. sidewalk is 50 per cent complete. Forty per cent of the cable wrapping is finished.

Modjeski, Masters, and Case, Inc., are the engineers, with W. C. Gorman acting as resident engineer. The Kansas City Bridge Co. built the substructure at a contract price of $357,601, with I. E. Hayes acting as superintendent. The McClintic-Marshall Corp. is doing the super-structure work with L. L. Martin as resident engineer and A. Toohey as superintendent. The contract is for $686,798.

When the bridge is completed it will be the first suspension to span the Mississippi River below Prairie Du Chien, Wis.

Alternate Plan For Highway Work Allows Higher Man-Year Cost On Projects

STATES financially unable to underwrite their entire allotment of federal funds for highway construction are permitted by the Bureau of Public Roads to underwrite a portion of them in order to include a higher type of work than would otherwise be possible.

Instructions were issued this week for applying in this way the alternate plan set up in the Bureau's regulations under which the state agrees to give employment to the full quota of labor that would be employed by its federal allotment at a rate of $1,400 per man-year in lieu of striking this average on each highway project. The $1,400 man-year limitation has been removed in its entirety with respect to grade crossing elimination projects and the alternate plan as applied to such work consequently has likewise been stricken out, as reported in ENR. Sept. 12.

The effect of the modified man-year regulation on highway construction is that a state may set up any portion of its total program and with respect to that portion ignore the $1,400 limitation by adding enough funds of its own so as to produce employment equivalent to dividing by $1,400 the federal allotment for such portion of the state's total program. On other portions of its program the state must adhere to the $1,400 limitation if it has no more money of its own to supplement federal funds. Each state's total allotment of federal funds must produce employment at a rate of $1,400 per man-year. No state is compelled to draw down its total allotment if it does not wish to. The purpose of this broad interpretation of the man-year limitation was to enable states which do not have enough funds to underwrite federal funds on their entire program to include some higher man-year cost projects.

Contract modification clause omitted

Section 10 of the highway and grade crossing regulations was amended this week by striking out the provision subjecting contracts to modification in accordance with any subsequent legislation that may require observance of minimum wages or maximum hours for labor employed in the production of materials purchased by the contractor.

Equipment Manufacturers Plan Association

A group of construction equipment manufacturers, feeling the need for an association composed of members of existing trade associations and groups of manufacturers, met in Chicago on July 17 and adopted the name "Construction Equipment Association." An organizing committee has been appointed.

TAYLOR PARK DAM SITE FOR UNCOMPAHGRE PROJECT

Reservoir site and location of Taylor Park Dam, near Gunnison, Colo., being constructed by the Bureau of Reclamation. An allotment of $2,000,000 has been made for the construction of an earthfill dam here to supplement the water supply available for the Uncompahgre irrigation project around Montrose and Delta. A white line on the canyon wall in the left foreground marks the top of the dam. The diversion tunnel was holed through in August.

In the background may be seen the Collegiate Range. The foothills of this range will form the border of the reservoir.

Court Holds Contractors Liable in Activated Sludge Case

Ten contractors furnishing equipment for the $1,800,000 extension to the Milwaukee Jones Island sewage treatment plant were held, on Sept. 5, to be contributing infringers of the patents held by Activated Sludge, Inc. Contracts with the contractors provided that the Milwaukee Sewerage Commission would assume any responsibility of suits and intervened when suit was brought against the contractors for contributory infringement.

In addition to the activated sludge patents which were held infringed by the Milwaukee Sewerage Commission in a former suit this suit included the two other patents passed on in the case of the Sanitary District of Chicago. These were a general process patent which Judge Lindley in the Chicago case held valid and infringed and a reissue of the spiral flow patent, the original of which Judge Lindley stated included too much and failed to show invention. The reissued patent with fewer claims Judge Geiger held valid and infringed.

The contractors will have to sue the commission for whatever damages may be assessed against them.

Fort Peck Dredging Operations Exceed 3,600,000 Yd.

The four hydraulic dredges pumping fill into the Fort Peck Dam placed 3,625,000 cu.yd. of material in the embankment during the month of August, exceeding the designed capacity of the dredging plant by 22 per cent. This yardage marks the monthly record of fill placing to date. The rate of operations is being stepped up in September, for in the first ten days of this month 1,243,560 cu.yd. of material was pumped, as compared with 1,209,000 for the same period in August.

In the four 32-ft. diversion tunnels, where enlarging from a center pilot heading to full size is under way (*ENR*, Aug. 29, 1935, p. 296) the Mason & Walsh Co. is now resorting to the heading and bench method of enlargement on three of the bores, retaining the former full-section method on tunnel No. 3 for the present. The change in method has speeded up the work of enlarging. All muck from the top headings is taken out through the pilot tunnels to the upper portals. The face of the top heading is carried from 50 to 75 ft. ahead of the bench. Following close behind the bench excavation is the placing of invert concrete, followed in turn by completion of the sidewalls and arch.

State and Federal Examinations For Engineers to Be Held Soon

Civil service examinations for positions as senior valuation engineer, assistant valuation engineer and associate valuation engineer in the New York Department of Public Service will be held by New York State on Oct. 5, 1935. Copies of the announcement and applications may be secured from the Department of Civil Service at Albany.

Applications for examination for senior safety engineer in the Division of Labor Standards of the U. S. Department of Labor must be filed in Washington not later than Sept. 30, 1935. The necessary forms may be obtained from any first class post office.

Interstate Board Proposed To Control Ohio River Pollution

An interstate commission, representing seven states, to control the amount of pollution entering the Ohio River was proposed on Sept. 10 by the committee on stream pollution of the Cincinnati Chamber of Commerce. The commission would be composed, under the plan, of from one to five representatives of each of the states of Ohio, Kentucky, Indiana, Pennsylvania, West Virginia, Illinois and Tennessee. The number of members from each state would depend upon the proportion of funds contributed. It is proposed that the funds be raised from a small tax on waterworks.

The proposal will be taken up with the National Resources Board for approval and suggestions. Congressional approval is necessary before such a drainage authority can be established. Similar authority was recently granted to New York, New Jersey and Connecticut, and the Tri-State Board that was set up at that time has already held its preliminary meeting.

Personals

J. M. BELKNAP and SAMUEL M. BAILEY have been promoted to the positions of area engineers in the Muskingum watershed conservancy district.

OLIVER W. CROMWELL, IERA project engineer, Benewah County, Idaho, has resigned to accept a position with the state highway department.

ARTHUR L. TAYLOR, city consulting engineer in charge of the Beverly Hills, Calif., water system, has been appointed a member of the board of directors of the Metropolitan Water District of Southern California.

PROF. S. S. STEINBERG of the department of civil engineering, University of Maryland, will serve as president of the educational division of the American Road Builders' Association. Prof. Steinberg is a member of the highway committee of the Maryland planning commission.

LIEUT. COL. C. L. HALL, Division Engineer, Corps of Engineers, in charge of the Cincinnati district, has been selected to represent the War Department at the International Congress of Navigation, Brussels, Belgium.

LEONARD G. HOGUE, who has been engaged in sewage disposal and levee work with the Memphis district, Corps of Engineers on the Mississippi and St. Francis Rivers, has been made associate superintendent and has been given an assignment at Los Angeles, Calif.

PAUL L. MONTGOMERY, who has been engineer on the staff of the Pike County, Ohio, relief administration since January, 1934, has been transferred to the engineering staff of the WPA district No. 11, Chillicothe.

WILLIAM DARBY has been appointed chief engineer of the city power plant at Edmonton, Alberta.

WADE HAMPTON FOOTE has resigned his position as county surveyor, Coryell County, Tex., and at present is stationed in New York City on special investigation work for the U. S. Government, including technical and financial research.

Obituary

ROBERT P. PEEBLES, chief engineer for the Castalia Portland Cement Co., Castalia, Ohio, for the past seventeen years, died on Sept. 4.

FREDERICK J. GALOW, construction supervisor for the New York Telephone Co. in eastern Long Island, died at his home in Riverhead, L. I., on Sept. 10. He was 56 years of age.

LOUIS G. CARPENTER, well known irrigation engineer, died in Denver, Colo., Sept. 4, at the age of 74. Mr. Carpenter received his engineering education at Michigan Agricultural College, University of Michigan, and Johns Hopkins University. In 1888 he became professor of civil and irrigation engineering at Colorado Agricultural College, a position which he held for a number of years, serving simultaneously as Colorado state engineer from 1903 to 1905. Mr. Carpenter studied dams and irrigation work in France, Italy and Algiers and also undertook many assignments as consulting engineer.

DWIGHT FOSTER KILGOUR, for 50 years a consulting engineer at Boston, Mass., died at his home in Lexington, Mass., on Sept. 10, after a short illness. He was born at Lynn, Mass., in 1863 and devoted much attention to building construction. For a time he was water and sewer commissioner of Lexington.

JULIAN WAY KENDRICK, 75, died in Birmingham, Ala., Sept. 4 after a protracted illness. Mr. Kendrick was actively engaged as a consulting engineer up to the time of his death. He was born at Front Royal, Va., graduated from the Virginia Polytechnic Institute, and came to Birmingham about 1887, and was associated with engineers in laying out what is now the site of Birmingham. He was at one time engaged in the coal mining industry, being superintendent and engineer at the Helena Mines. He served as city engineer of Birmingham from 1891 to 1907 and again from 1913 to 1921, and directed extensive programs of grade separation and the construction of sewerage systems for the city, and designed and constructed the original trunk sewerage system and disposal plants for Jefferson County, while serving as sanitary engineer.

CONSTRUCTION STATISTICS OF THE WEEK

PUBLIC engineering construction awards dropped 30 per cent this week with the loss distributed between waterworks, sewerage and public buildings. Total awards for the week are $24,746,000 of which private work is $4,159,000, an increase over the preceding week. Public awards are down to $20,587,000 of which $7,779,000 is federal and $12,808,000 state and municipal, both considerably below last week. Corresponding figures last year are: total, $25,187,000; private, $5,784,000; public, $19,403,000; federal $3,747,000; state and municipal, $15,656,000.

Highway awards are higher at $7,164,000 and earthwork, drainage and waterways are higher at $5,950,000. Commercial building are slightly lower at $1,555,000, industrial buildings at $1,765,000 are about the same as last week; public buildings at $2,280,000 are down as are waterworks at $1,035,000 and sewerage at $908,000. Bridge awards at $1,656,000 are approximately the same as last week.

The larger projects include: annealing furnace for Bethlehem Steel Corp., Sparrows Point, Md., $500,000; elevators for Dept. of Interior Bldg., Washington, D. C., $528,000; mail handling equipment, elevator plant and lighting fixtures, Post Office, St. Louis, Mo., $505,000. Highway awards include: $750,000 by Iowa; $842,000 by Maine; $2,207,000 by New York; $570,000 by Michigan; $576,000 by Wyoming. Federal awards include dredging Chesapeake & Delaware canal between Bethel and Court House Point, 3 contracts, $2,191,000; relocation, B. & O. R. R. at Tygart River Reservoir, Grafton, W. Va., $2,935,000. In New York City, the Dept. of Parks decided to go ahead with the stadium on Randall's Island using work relief labor, $1,000,000.

New capital for the week includes $11,501,000 in state and municipal bond sales, $1,000,000 in corporate securities sales. Federal allotments include $24,750,000 for slum clearance and low cost housing.

CONTRACTS

(Thousands of Dollars)

	Weekly Average Sept. 1934	Prev. 4 Weeks	Week Sept. 19 1935
Federal Government	$4,256	$11,590	$7,779
State and municipal	15,021	14,252	12,808
Total public	$19,277	$25,842	$20,587
Total private	4,333	3,837	4,159
Week's total	$23,610	$29,679	$24,746
Cumulative to date:			
1934..$1,007,923,000	1935..$989,286,000		

NEW PRODUCTIVE CAPITAL

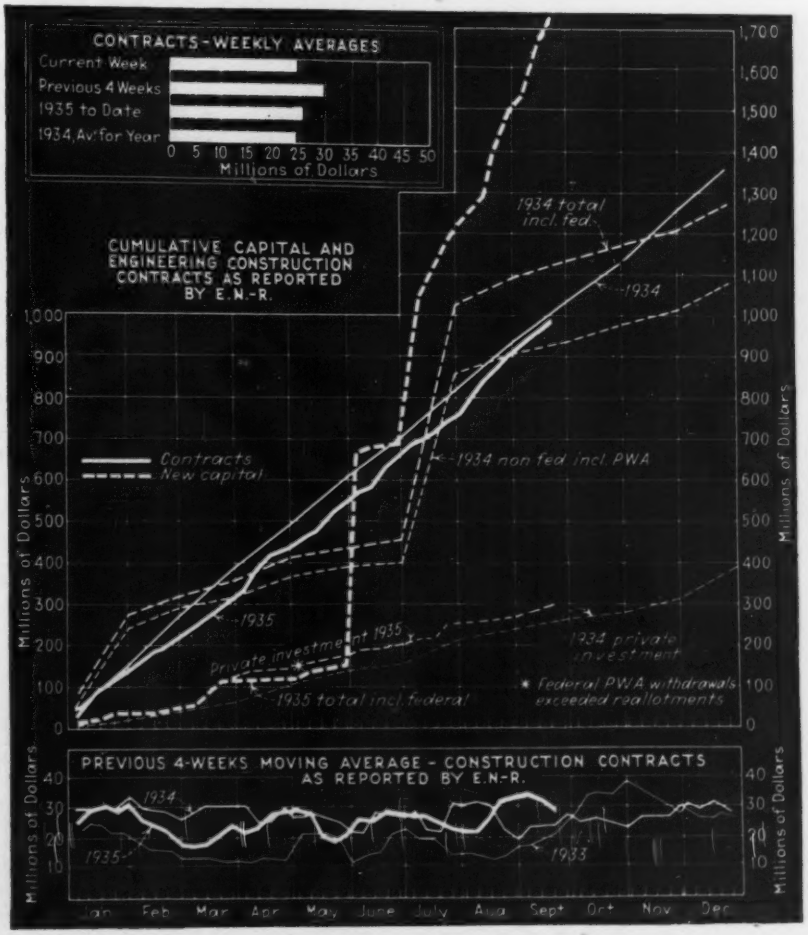
(Thousands of Dollars)

1935	Week Sept. 19	Cumulative
State and municipal	$11,501	$298,127
PWA allotments, S&M.	.....	402,829
RFC loans, S&M.	.....	36,084
Corporate issues	1,000	22,989
PWA private	.....	8,629
Local contrib. to WPA.	2,249	14,030
Total non-federal	$14,750	$782,688
PWA federal	42,113	941,308
Total new capital	$56,863	$1,723,996
Cumulative to date:		
1934..$1,117,437,000	1935..$1,723,996,000	

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for federal construction, deficiency act funds. An arbitrary percentage (25%) of the WPA allotments and local contributions to WPA work is included to allow for the capital additions through the Works Progress Administration division of the new program.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Sept., 1935	195.10	93.82	Aug., 1935	140	61
Aug., 1935	195.10	93.82	July, 1935	96	42
Sept., 1934	200.58	96.41	Aug., 1934	94	41
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56



Construction Equipment and Materials

New Model Concrete Vibrator Designed for Easy Handling

A flexible electric concrete vibrator, known as Model No. 15, has recently been added to the line of vibrators manufactured by the Viber Co., Los Angeles, Calif. This new model is equipped with a Viber double tube construction vibrator 1¾ in. in diameter and 18 in. long. It is equipped with a ½ hp., 110 v. motor for a.c. or d.c. The motor, which weighs about 17 lb., develops 8,000 r.p.m. Model No. 15 has 12-ft. flexible shaft 1¾ in. in diameter for access to difficult spots. Although the complete assembly weighs 50 lb. the operator lifts less than 15 lb.

Portable Rock Crusher Developed by FWD

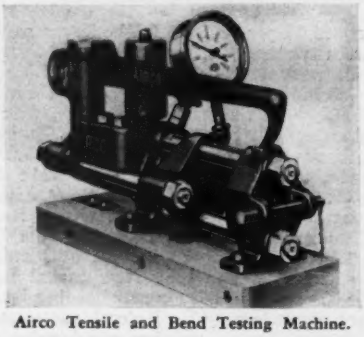
A new portable rock crusher unit has been developed by the Four Wheel Drive Auto Co., Clintonville, Wis. The new machine consists of a 3-ton FWD truck equipped with an overhead eccentric type rock crusher mounted on the rear of the frame and driven by the truck motor. The crusher is equipped with continuous bucket feeding and loading elevators. This portable crusher is designed to answer demands for a machine which can be speedily moved from one road building location to another.

The rock crusher is driven from the main shaft of the transmission. With a governed engine speed of 1,200 r.p.m. in direct drive and by means of a four to one reduction in the drive sprockets of the crusher, a crushing speed of 300 r.p.m. can be maintained. The rated capacity of the unit at this speed ranges from 50 to 125 tons per day, varying with the hardness and size of the stone and gravel used.

Portable Testing Machine For Welding Work

To enable engineers and welders to determine the strength of welds, while working in the field on fabrication or pipeline work, the Air Reduction Sales Co., New York City, has developed the Airco Portable Tensile and Bend Testing Machine.

The machine consists essentially of an oil pump, a hydraulically actuated piston, one fixed and one movable head, for gripping the test specimens and four symmetrically located steel shafts, two of which transmit the piston motion and load to the movable heads while the other two maintain the alignment of the two heads. With a few strokes of the operating lever, a direct load up to 40,000 lb. per sq.in. can be exerted on specimens of ½ in. in thickness and 1½ in. wide. By using specimens of smaller cross-sectional areas, loads equivalent to 150,000 lb. per sq.in. can be applied. Readings, which are made on a 6-in. gage calibrated against a standard dead-weight tester, are accurate to within 2 per cent. For bend testings the machine is provided with an anvil which fits into the movable head and which bends the specimen against a pair of supports.



Airco Tensile and Bend Testing Machine.

The testing machine, which weighs 200 lb., is mounted on a hardwood base which forms part of the packing case, sides and top of which are easily assembled, allowing it to be shipped quickly and easily. A separate box is provided to protect the gage while in transit.

New Publications

AERO-2-STAGE PORTABLE COMPRESSORS, Models 105, 160, 210 and 315, Worthington Pump & Machinery Corp., Harrison, N. J. 8½x11. Each bulletin 4 pages.

GEARMOTORS, Allis-Chalmers Manufacturing Co., Milwaukee, Wis. 8½x11, 4 pages. Leaflet 2203.

THE BIN-DICATOR, Ripley Manufacturing Co., Wayne, Mich., 3½x9½, 8 pages.

BROOKVILLE LOCOMOTIVES WITH FORD V-8 POWER, Brookville Locomotive Co., Brookville, Pa. 8½x11, 8 pages. Bulletin No. B V-8.

MARMON-HERRINGTON ALL WHEEL DRIVE FORD V-8, Marmon-Herrington Co., Inc., Indianapolis, Ind., 8½x11, 6 pages.

THERM-O-TILE STEAM CONDUIT SYSTEM SPECIFICATIONS, H. W. Porter & Co., Inc., Newark, N. J. 8½x11, 8 pages. Bulletin 352.

CINDER RECLAIMING AND FLY ASH DISPOSAL, United Conveyor Corp., Chicago, Ill. 8½x11, 16 pages.

AUDUBON WIRE CLOTH, Audubon Wire Cloth Corp., Philadelphia, Pa. 8½x11, 8 pages. Bulletin No. 42.

AIR CUSHION VALVE COMPRESSOR, Pennsylvania Pump & Compressor Co., Easton, Pa. 8½x11, 16 pages. Bulletin No. 164.

VICTOR-ACME ROTARY POSITIVE BLOWERS, Roots-Connersville Blower Corp., Connersville, Ind., 8½x11, 4 pages. Bulletin 21-B17.

ZEOLITES, MINING, PROCESSING, MANUFACTURE AND USES, Permutit Co., New York City. 5½x8½, 30 pages.

COLMONOY OVERLAY METAL, Colmonoy, Inc., Los Nietos, Calif. 3½x9, 16 pages.

FRANCKE FLEXIBLE COUPLINGS, John Waldron Corp., New Brunswick, N. J. 8½x11, 12 pages. Catalog No. 50.

HEATING FOR THE MODERN HOME, The Trane Co., La Crosse, Wis. 8½x11, 8 pages.

Business Notes

JAMES R. FITZPATRICK, until recently vice president in charge of sales of the Haskelite Co. has been appointed director of sales of the technical division of the Algoma Plywood and Veneer Co. with headquarters for sales, research and engineering service in Builders Building, Chicago. Recently this company installed one of the largest hot-plate presses for producing panels 12 ft. wide of any length and thickness.

J. G. HOAG, who has been connected with the Philadelphia office of the Osgood Co., has been appointed manager of the Philadelphia branch.

THE C. H. & E. MANUFACTURING CO., Milwaukee, Wis., has announced the appointment of the following as distributors of C. H. & E. saw rigs, pumps, hoists, and mortar mixers: The Albany Construction Equipment Co., Albany, N. Y.; Brown and Sites Co., New York City; The Paving Supply & Equipment Co., Washington, D. C.; H. B. Fuller Equipment Co., Cleveland, Ohio.

THE C. O. BARTLETT AND SNOW CO., manufacturers of material handling equipment, Cleveland, Ohio, has announced the appointment of W. A. Meddick to a position in the sales department.

THE KENNEDY VALVE MANUFACTURING CO., announces the appointment of Mr. Thomas C. Swope as their Texas and Louisiana representative, with headquarters in Beaumont, Texas.

PANGBORN CORPORATION, Hagerstown, Md., announces the election of P. J. Potter, former 2nd vice president, as a director and vice president of the corporation.

THE CATERPILLAR TRACTOR CO., Peoria, Ill., has secured the services of Mr. O. Q. Hinds, formerly road machinery supervisor for the Allis-Chalmers Manufacturing Co., as special road machinery representative.

LINK-BELT CO., Chicago, has appointed the Henry A. Petter Supply Co., Paducah, Ky., as distributors of Link-Belt crawler-mounted shovels, cranes and draglines.

INLAND STEEL CO., Chicago, announces the appointment of Charles R. Robinson, as vice president and general sales manager. Mr. Robinson succeeds Edward M. Adams.